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THE INFLUENCE OF STRAIN ON THE SHEAR
STRENGTH PARAMETERS OF A HIGHLY PLASTIC
PRECONSOLIDATED, HOMIONIC CLAY SOIL

by



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A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES
IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE OF
MASTER OF SCIENCE

DEPARTMENT OF CIVIL ENGINEERING

EDMONTON, ALBERTA

NOVEMBER 1967

UNIVERSITY OF ALBERTA
FACULTY OF GRADUATE STUDIES

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies for acceptance, a thesis entitled "THE INFLUENCE OF STRAIN ON THE SHEAR STRENGTH PARAMETERS OF A HIGHLY PLASTIC, PRECONSOLIDATED, HOMIONIC CLAY SOIL", submitted by GARRY ROBIN GILCHRIST in partial fulfilment of the requirements of the degree of Master of Science.

Date June 26, 1967

ABSTRACT

The purpose of this thesis was to extend previous research at the University of Alberta on the influence of strain on the shear strength parameters of remoulded, highly plastic, homionic, normally consolidated clays to that of the same soil in an overconsolidated state. The CFS (Cohesion-Friction-Strain) triaxial test was again employed. Three calcium-clay samples were tested at 1, 2 and 4 kg/sq.cm. with one potassium-clay and one magnesium-clay also being tested at 1 kg/sq.cm.. All the samples were originally preconsolidated at 8 kg/sq.cm.. A detailed description of the discussion of the laboratory procedure is included. The results are presented in graphical and tabular form to show the factors that effect cohesion and friction with special emphasis on the effect of overconsolidation.

For the soils tested, cohesion was shown to be dependent on both stress history and the cell pressure at the time of testing. The adsorbed cation appeared to have some effect on the mobilized cohesion at higher strains but the variations noted were within the range of normal test error, thus no definite trend can be presented. The percent axial strain at which peak cohesion occurred, approximately 1 to 2% for all samples, did not appear to be effected by system characteristics or stress history.

Mobilized frictional resistance for the soils tested was shown to be independent of both the cell pressure and the time of testing and the adsorbed cation. It was felt that the grain size distribution and type of clay particle present played an important role in the manifestation of mobilized frictional resistance. Pre-consolidation resulted in lower void ratios, thus larger peak angles of internal friction were developed. The strain at which peak friction was obtained decreased with increasing overconsolidation ratios.

It is recommended that investigations of the effects of various pore fluids on cohesion and friction, as well as more tests on overconsolidated samples by means of the CFS test technique be undertaken which may prove useful to the understanding of shear strength.

ACKNOWLEDGEMENTS

The author thanks Associate Professor S. Thomson for his thoughts and guidance which he directed towards this thesis. For his untiring efforts and deep interest the author is deeply indebted.

Sincere thanks are extended to the National Research Council of Canada who provided the financial assistance necessary to make this program possible.

To the numerous others who provided assistance, many thanks.

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CHAPTER I

INTRODUCTION

1.1 The study of fine grained soils on a microscopic scale is an area relatively new to the engineer. The background of knowledge one must possess, to acquire and apply significant results to practical problems, is beyond the scope of the average engineer. Thus this area of study is approached with reluctance and some skepticism. Co-operation between soils engineers, soils scientists, colloidal chemists and geologists is required if significant advancement of the acceptance and contribution of this area of study is to be made.

1.2 It has been shown by many researchers that the nature of the adsorbed cation, associated with electrical requirements for neutrality, and high specific surface affects various engineering properties of fine grained soils. For this reason, the soils engineer is interested in the microscopic aspect of soils, particularly the effects on the shear strength and volume change characteristics.

1.3 P.A. Thomson, in studies at the University of Alberta (1960)*, investigated the effects of exchangeable cations on some of the properties of a clay shale. The presence of more than one exchangeable cation in significant quantities made interpretation of results difficult, hence the need for homionic soils was established. Homionic soils were

*References listed alphabetically in "List of References"

prepared by Hamilton (1961) for the purpose of studying the effects of different adsorbed cations on consolidation characteristics, liquid limits and plastic limits. It was found that an homionic soil could be prepared and results from the above tests were dependent upon the adsorbed cation. Further investigations by S. Thomson (1963) and J.G. Locker (1963) showed variations in shear strength with different adsorbed cations and varying pore water salt concentrations. These shear strength tests were consolidated undrained tests with pore pressure measurements. In 1960, J.H. Schmertmann and J.O. Osterberg, presented a new triaxial shear strength test technique referred to as the CFS test (Cohesion-Friction-Strain). The procedure involves alternating between two constant preselected values of major principal effective stress (σ'_1). Thus two stress-strain curves are obtained from one sample. This enables one to obtain the mobilized cohesion and friction at any strain. This procedure was employed by Dahlman (1965) on a normally consolidated, homionic, remoulded, highly plastic clay soil. The conclusions from this research were: cohesion is dependent on the consolidation pressure and independent of the type of adsorbed cation; the angle of internal friction is related to the type of adsorbed cation and independent of the consolidation pressure.

1.4 Dahlman's work was restricted in scope to normally consolidated soils, hence his results have questionable applicability to overconsolidated materials. Since the latter type of soils are prevalent in Western Canada and also to present a more complete picture, it was considered desirable to test these same soil modifi-

cations in an overconsolidated state. Thus the purpose of this research topic was to determine the effect on the mobilized components of shear strength by preconsolidating the clay soil. The same soil with a similar modification procedure to that of Dahlman's (1965) was used in order that results could be compared. Four different soil types were prepared which consisted of a calcium-clay, potassium-clay, magnesium-clay, and sodium-clay. Due to the limited time available to the author and the very long times needed for consolidation and rebound of the sodium saturated samples, tests were not performed on this modification. Three tests at overconsolidated ratios of 2, 4 and 8 were performed on the calcium-clay. Two tests, one on each of the potassium-clay and magnesium-clay were run, both at an overconsolidation ratio of 8. All the samples were preconsolidated at 8 kg/sq.cm. initially, then rebounded to the appropriate pressures.

1.5 Shear strength values were obtained by means of triaxial tests employing the CFS test technique. Due to the limited number of samples tested, the results will be more of a qualitative nature than quantitative.

1.6 A detailed discussion of soil physics and clay mineralogy is not included. For detailed information other literature is recommended, for example Grim (1953, 1962) or Baver (1956).

1.7 The following chapters present a brief review of pertinent literature, sample preparation and discussion, and examination of the test technique. The results of the testing program are presented and

discussed with a physical-chemical interpretation being employed. The conclusions are then drawn in light of the above discussion, followed by recommendations for further research.

CHAPTER II

LITERATURE REVIEW

Basic Shear Strength Theories

2.1 The oldest and most widely used statement of shear strength is that expressed by the Mohr-Coulomb failure criterion (Taylor, 1948).

This may be written in the form:

$$\tau_f = c + \sigma_f (\tan \phi)$$

where: τ_f = shear strength at failure

c = cohesion intercept

σ_f = normal stress on the failure surface

ϕ = angle of internal friction.

This equation is relatively simple but is limited in general use because pore water pressure is not considered. Application of Terzaghi's fundamental concept of neutral and effective stress to the Mohr-Coulomb equation yields the following:

$$\sigma'_f = \sigma_f - \mu$$

and: $\tau_f = c' + \sigma'_f (\tan \phi)$

where: σ'_f = effective normal stress on the failure surface

μ = pore water pressure on the failure surface

c' = effective cohesion

ϕ' = effective angle of internal friction.

The above terms, c' and ϕ' , are called effective stress parameters. A further elaboration of the basic Coulomb equation results in the Krey-Tiedemann shear strength concept (Tschebotarioff, 1951, page 155). In terms of effective stress the Krey-Tiedemann equation may be expressed as follows:

$$\tau_f = \sigma'_p \tan \phi'_c + \sigma'_f \tan \phi'_r$$

where: $\sigma'_p \tan \phi'_c$ = cohesion intercept

σ'_p = effective preconsolidation pressure

ϕ'_c = angle of shear strength related to cohesion

$\sigma'_f \tan \phi'_r$ = that component of τ_f due to internal shear resistance

σ'_f = present normal stress

ϕ'_r = angle of shear strength associated with rebound.

The values of c' and ϕ'_r are not considered to be the actual cohesion and angle of internal friction but mathematical components.

The CFS Test

2.2 The laboratory test employed in this research, referred to as the Cohesion-Friction-Strain (CFS) test, was presented by Schmertmann and Osterberg in the Boulder Conference on the Shear Strength of Cohesive Soils (1960). The CFS test enables one to obtain more than one (generally two) stress-strain curve from a single sample by varying the pore pressure to maintain a constant pre-selected value of σ'_1 . After sufficient data has been gathered to define the stress-strain curve on a plot of deviator stress versus strain, the pore pressure is varied

to maintain a new pre-selected value of σ'_1 . Data are then obtained at this value of σ'_1 . This "hopping" between pre-selected curves is continued past the maximum deviator stress. The test allows the determination of the strain-mobilized components of cohesion and friction of the resistance of a soil to shear stress from one soil sample. The components were defined by Schmertman and Osterberg (1960) in the following manner:

Cohesion (c_e) - "The cohesion of a soil at any strain, is the shear stress developed on the plane of Mohr envelope tangency at that strain, if the intergranular stress on that plane could be reduced to zero without significant change in soil structure."

Angle of Internal Friction (ϕ_e) - "The angle of internal friction, at any strain, is the angle whose tangent is the ratio of the change in shear stress to the change in normal intergranular stress occurring on the plane of Mohr envelope tangency at that strain during a stress change occurring without significant change in soil structure."

2.3 To calculate c_e and ϕ_e the following expressions, as derived from the Coulomb-Hvorslev failure equation, are used:

$$\phi_e = \sin^{-1} \frac{\Delta\sigma_d}{2(\Delta\sigma'_1) - \Delta\sigma_d}$$

and:
$$c_e = \frac{\sigma_d/2 - (\sigma'_1 - \sigma_d/2) \sin \phi_e}{\cos \phi_e}$$

where: $\sigma_d = (\sigma'_1 - \sigma'_3) = \text{deviator stress.}$

Physical-Chemical Phenomena

2.4 The many unknowns and variables in this area of soil mechanics, particularly as it applies to shear strength, leads one to discussing trends rather than specific data to which numerical values can be attached.

2.5 Lambe (1958) states:

"Any change in the soil-water system which expands the double layer tends to decrease soil strength (at a given void ratio), since interference by the double layers of the two adjacent colloids increases interparticle repulsion. This concept leads to the prediction that any of the following changes would generally reduce the shear strength of a clay:

1. Reduction of electrolyte concentration
2. Cation exchange from high to low valence (e.g. Ca^{++} to Na^+)
3. Exchange from a cation of small hydrated radius to a cation of large hydrated radius (e.g. Na^+ to Li^+)
4. Adsorption of anions (e.g. phosphate)
5. Increase in dielectric constant of pore fluid
6. Increase in pH of pore fluid
7. Decrease of temperature
8. Increase in water content."

As was stated previously (paragraph 1.4), the only intended variable in this research program, other than moisture content, was the adsorbed cation. TABLE II.I lists the various characteristics of the four cations employed in this program. However, it is to be noted that there does not appear to be any particular correlation between the radius of the hydrated soil particle and that of the hydrated cation. Further, there is some doubt as to whether or not the sodium cation is hydrated when adsorbed (Grim and Cuthbert, 1945).

TABLE II.I
CHARACTERISTICS OF VARIOUS CATIONS

Cation	Calcium	Magnesium	Potassium	Sodium
Non Hydrated radius ° 1 Å	(1.17) (1.06)	(0.89) (0.78)	1.33	0.98
Hydrated radius Å ° 1	9.6	10.8	(5.32) (3.8)	(7.9) (5.6)
Zeta Potential of adsorbed cation ° 2	52.7	54	56.5	57.7
Nature of oriented water generated by adsorbed cation ° 3	well oriented to approx. 10 Å and transition to non-oriented abrupt	well oriented to slightly less than Ca	forms tight bond between particles, thus very thin oriented layer and well defined boundary	well oriented to approx. 7.5 Å and gradual transition to tens of mole- cular layers
Soil Structure Tendency ⁴ when cation is adsorbed	flocculated	flocculated	both dispersed and flocculated	dispersed
General Order of Replaceability ° 5	1,2	2,1	3	4 (most easily replaced)
Polarizability, cm ³ ° 6	--	--	.987 x 10 ⁻²⁴	.504 x 10 ⁻²⁴
Atomic No.	20	12	19	11

1. Grim (1953) p. 148
2. Bauer (1956)
3. Grim (1958) p. 20
(1953) p. 174
4. Lambe (1958)
5. Grim (1953 p. 145)
6. Rosenqvist (1955)

CHAPTER III

SAMPLE PREPARATION AND THE CFS TRIAXIAL TEST

Soil Used in Program

3.1 The soil used in this program was from a high cut in Belgravia Ravine, Edmonton, Alberta. The highway cut was opened in the latter part of the summer of 1959 with soil samples being taken in early April, 1960. After stripping some 6 to 8 inches of surface material, about 500 pounds of soil were dug out at a depth of approximately 15 to 20 feet below the original ground surface. The soil was air-dried and stored in the laboratory in a large, covered, shallow container. The material is a glacial lake sediment and is classified as a highly plastic clay. The same material has been employed in research work by Hamilton (1961), Thomson (1963), Locker (1963), and Dahlman (1965).

3.2 Classification tests in accordance with ASTM procedures yielded the results shown in TABLE III.I. Results from the Alberta Research Council of mineralogical composition of the clay size fraction as determined by X-ray diffraction are also included in the table.

Soil Modifications

3.3 The purpose of modifying the original material obtained in the field was to yield four homionic soils with essentially zero salts in the pore water. Modifications to obtain a sodium-clay, potassium-clay

TABLE III.1¹

SUMMARY OF CLASSIFICATION TESTS ON ORIGINAL CLAY

TEST	RESULT
SPECIFIC GRAVITY	2.79
ATTERBERG LIMITS	
LIQUID LIMIT	74.3
PLASTIC LIMIT	30.8
PLASTICITY INDEX	43.5
GRAIN SIZE DISTRIBUTION ²	
% SAND SIZES	4
% SILT SIZES	34
% CLAY SIZES	62
MINERALOGICAL COMPOSITION ³	
MONTMORILLONITE	30 – 40 %
ILLITE	30 – 40 %
CHLORITE	20 – 30 %

¹ FROM SEMCHUK [1962, P. 15] WITH PERMISSION

² M. I. T. GRAIN SIZE SCALE

³ AS DETERMINED BY RESEARCH COUNCIL OF ALBERTA
ON CLAY SIZE FRACTION - APPROXIMATION.

calcium-clay and magnesium-clay were performed. The following procedure is based on the work of Thomson (1963), Locker (1963), and Dahlman (1965).

3.4 Air-dried soil was mechanically broken down to pass a No. 40 sieve by means of a mortar and pestle. Three thousand gram batches were weighed and placed in ten-quart polyethylene buckets. Six thousand millilitres of 1.0 N ammonium acetate solution was added, and the mixture was stirred for approximately 1 hour with a large paddle mixer. After allowing the mixture to stand for at least 24 hours or until the soil had settled, approximately three thousand millilitres of clear supernatant solution were siphoned off and immediately replaced by three thousand millilitres of 1.0 N ammonium acetate which was then stirred for 2 hours. The above procedure was repeated three more times, resulting in eighteen thousand millilitres of 1.0 N ammonium acetate being used over a ten day period. Immediately after the last ammonium acetate wash, the acid wash was commenced. The above procedure was repeated using six washes, each consisting of three thousand millilitres of 0.75 N hydrochloric acid (HCl). After the last acid wash, the soil was washed three times with three thousand millilitres of distilled water to reduce the concentration of the remaining acid. Following the last water wash, the mixture was stirred and split into four approximately equal volumes with each being placed into a ten-quart polyethylene bucket. It was assumed that the above applications of ammonium acetate and hydrochloric acid were sufficient to remove any carbonates and sulphates. In each of the above four buckets a soil modification to a homionic soil was performed. Three thousand millilitres of 1.0 N aqueous

ammonium acetate salt solution of the desired cation were added to each bucket of soil. The contents were mixed for 2 hours and then allowed to settle for a minimum of 24 hours. The supernatant liquid was siphoned off and another wash of four thousand millilitres of 1.0 N acetate solution was added. This procedure was repeated once more resulting in an acetate to soil ratio of about 15 to 1 (on the basis of weight). After the last acetate wash, the supernatant liquid was siphoned off and the alcohol was begun. Four thousand millilitres of chemically pure ethyl alcohol were added to each of the four soil types after which stirring continued for 2 hours. The mixture was allowed to stand for a minimum of 24 hours. The supernatant liquid was then siphoned off and four thousand millilitres of fresh alcohol were added, the mixture stirred and allowed to stand once again. It was felt that two alcohol washings were sufficient for all the soil types except the calcium-clay. One more wash of three thousand millilitres of ethyl alcohol was added in this case. At this time small samples were removed from each bucket and cation exchange capacity results were obtained. This analysis showed the soils to be homionic, but the pore water had a high concentration of salts in all cases except the sodium-clay. Upon personal consultation with Dr. Pawluk, Department of Soil Science, it was decided to dilute the concentration of salt in the pore water by washing with a 0.001 N acetate solution of the desired cation. With the pore salts having a much lower concentration, it was believed the alcohol could dissolve and remove the acetate ions much more easily. Dilute acetate washes of the desired cation were begun on all the soil types except sodium. Five thousand millilitres of 0.001 N acetate solution were

added, stirred and allowed to stand for at least 24 hours. This procedure was continued until the potassium-clay had been washed once more, the magnesium-clay three times more, and the calcium-clay five times more. At this time the alcohol washings were once again resumed. Five thousand millilitres of ethyl alcohol were added to each bucket except sodium. It was felt that this would be sufficient for the magnesium-clay and potassium-clay. Since calcium acetate is less soluble, two additional washings of five thousand and four thousand millilitres of ethyl alcohol, respectively, were employed. It was assumed at this time that the treatment was sufficient to remove the excess salts. Each soil type was then placed in an evaporating dish and dried at 50° C. The dried soil was crushed by means of a mortar and pestle to pass a No. 40 sieve. The soil was mixed to insure uniformity and a sample of approximately 30 grams was removed from the calcium, magnesium, and potassium clays for cation exchange capacity analysis. The remaining soil, about 600 grams, was split into three equal portions, each portion thus obtained being sufficient to make one triaxial specimen.

Moulding Triaxial Specimens

3.5 Each of the above portions of modified soil was then mixed with sufficient distilled water to form a thick slurry at a moisture content about 5% above the liquid limit of the soil. After a uniform slurry was obtained, the pan containing the soil was placed in a plastic bag and stored in the moisture room for about 24 hours. Prior to use, the soil was remixed. The specimens were moulded using a one-dimensional consolidation apparatus. The apparatus consisted of an inner lucite

tube, 1 3/4 inches in diameter and 5 3/8 inches long, in which four vertical, 1/2 inch wide, saturated, Whatman Number 54 filter paper strips were placed and wedged at the bottom by means of a porous stone covered by two pieces of filter paper. The slurry was placed in the lucite tube with the aid of a spatula, then the lucite tube plus soil was tapped on a hard surface to remove entrapped air after each addition of soil. A loading cap with drainage aids and filter paper was placed in the lucite tube and on top of the soil when the lucite tube was filled to within 1/2 inch of the top. This entire assemblage was placed in a 2 3/4 inch diameter lucite container with the annular volume between the cylinders being filled with distilled water. Immediately a lever arm was brought into contact with the load cap, a dial gauge was adjusted to measure deflections, and static weights were applied resulting in a stress of approximately 0.35 kg/sq.cm. on the specimen. At the same instant that the static load was applied, a timer was started in order that readings of deflection versus log time could be obtained and plotted. Hence, in this way, the progress of consolidation was observed. Three load increments were used, 0.35, 0.65, and 1.30 kg/sq.cm., with full primary consolidation being allowed under each increment. Applying the load in steps reduced any tendency for the soil to squeeze by the porous disc and loading cap. Under the final load, consolidation was allowed to continue until at least theoretical 100% consolidation had occurred as observed on the deflection-log time plot. At this time the apparatus was dismantled, the specimen extruded, wrapped in polyethylene plastic and stored in the moisture room until required.

Mounting the Triaxial Test Specimen

3.6 The procedure followed in trimming and mounting the specimen was similar to that described in the Norwegian Geotechnical Institute Publication No. 21 (Andresen et al, 1957). A detailed description of the procedure followed is given by Locker (1963) and will be only summarized here. The specimen was trimmed to approximately 35.7 mm. in diameter by 80.0 mm. in length using the Geonor trimming apparatus. Five saturated wool wicks were inserted longitudinally into the specimen in a symmetrical pattern following which diameter, length and weight measurements were taken. The specimen was wrapped in saturated, slotted filter paper and mounted in the triaxial cell on a porous disc protected by a piece of filter paper. Two rubber membranes and two thin layers of silicone grease were placed around the specimen. The rubber membranes were secured to the load cap and pedestal with four O-rings. The triaxial cell was filled with distilled deaired water and topped with an oil seal about one-half inch thick. A confining pressure was applied and the sample allowed to expell water into a 25 milliliter stopcock burette. A single load increment was used for consolidation in all cases, with time and burette readings being taken. When secondary compression time was about double primary compression time, the confining pressure was lowered in one step to the desired test pressure, thus allowing rebound. Similar to consolidation, time and burette readings were taken during rebound. When rebound ceased or the change with time was very small, the burette was removed and the triaxial cell was mounted in the loading press.

The Triaxial Test

3.7 A detailed description of the procedures followed during back-pressuring, the pore pressure reaction test and the CFS test is given by Dahlman (1965) and hence is only summarized here. However, modifications to the above procedure are included in the following summary. The layout of triaxial equipment was as described in Method 2 (Dahlman, 1965), with the exception of pore pressure being measured by means of a pore pressure transducer instead of a mercury manometer. Back pressuring was accomplished by increasing the cell pressure and back pressure simultaneously by 2 kg/sq.cm. and allowing the system to stand for at least 24 hours. The standard pore pressure reaction test was accomplished by increasing the cell pressure by 1 kg/sq.cm. and measuring the build up of pore pressure over a time interval of 10 minutes. The cell pressure was then reduced by 1 kg/sq.cm. and the pore pressure decrease was measured over the same interval of time. The sample was allowed to stand under the back pressure for at least another 24 hours before testing. The CFS test started similar to an undrained triaxial test with pore pressure measurements at a constant rate of strain. Once the piston was seated the pore pressure was varied to obtain a constant pre-selected value of major principal stress (σ'_1). After sufficient data was obtained at this σ'_1 , the pore pressure was changed to maintain a new constant value of σ'_1 . The high and low values selected for all the tests were 1.20 and 1.00 times the test pressure, respectively. Data obtained during the test included pore pressure, load cell readings, volume change, strain and time. A plot

of deviator stress versus strain was kept as the test progressed. All tests were continued beyond the maximum deviator stress. At the end of the test all water lines leading to the sample were closed and the apparatus was dismantled. Measurements taken at the end of the test were the final wet weight, volume (by mercury immersion), moisture content and slope of the shear plane. Typical data sheets and sample calculations for a CFS triaxial test are given in APPENDIX B.

CHAPTER IV

DISCUSSION OF SAMPLE PREPARATION AND TEST PROCEDURES

Soil Used in Program

4.1 The basic or unprepared soil used in this program was chosen because of its availability, the fact that other research has been carried out in the physical-chemical field on the same soil, the relatively high clay mineral content, and the large number of classification tests that are available.

Soil Modification

4.2 The purpose of soil modification was to obtain a homionic soil with essentially zero salts in the pore water and without significant damage to the clay particles. The efficiency of the modification process can be assessed by the cation exchange analysis results presented in TABLE IV.I. If soils can be obtained with the only variable being the adsorbed cation, presumably the physical effect of the cation can be obtained if identical test procedure is adopted. This implies the samples be consolidated to similar moisture contents for comparative purposes. The effect of foreign cations will be small if present in sufficiently small quantities, but should be considered in analyzing results (Hamilton, 1961). If salt is present in the pore water, the shear strength of the specimen will be effected. Work performed by

TABLE IV.1
RESULTS OF CATION EXCHANGE ANALYSES

SOIL TYPE	CATION EXCHANGE CAPACITY ME/100 GMS A.D.S.	TOTAL CATIONS PRESENT ME/100 GMS A.D.S.	CALCIUM	MAGNESIUM	POTASSIUM	SODIUM	SALT CONTENT ME/100 GMS A.D.S.
CALCIUM	30.1	31.0	30.1	-----	0.8	0.1	0.9
MAGNESIUM	29.8	25.4	0.5	24.5	0.3	0.1	-----
POTASSIUM	27.5	30.8	0.9	-----	29.8	0.1	3.3
SODIUM	31.7	32.3	-----	-----	0.3	32.0	0.6

Locker (1963) showed the effect of varying pore-water salt contents on shear strength. Since the sodium cation is associated with a relatively thick double layer, the presence of salt in the pore-water is more critical in this modification than in the others. Although no specific numerical variations in shear strength parameters can be related to the amount of pore-water salts at this time (1967), the general trend of the above conditions can be helpful in analyzing the results.

4.3 Three thousand grams of air-dried salt, mechanically ground to pass a No. 40 sieve, were placed in a ten-quart bucket. The reason for placing the entire batch of soil in one bucket was to obtain identical soil modification treatment until the separate cation modification was performed. Also, it was felt that flooding the soil with a solution and stirring it slowly for a long period of time would result in less damage to the clay particles, better removal of salts and a more complete adsorption of cations. The percent loss of soil due to mixing, slopping, etc., was less when working with a large batch of soil.

4.4 The procedure finally adopted for the first stage of modification to the soil was five washings with 1.0 N ammonium acetate and six washings with 0.75 N hydrochloric acid. The purpose of these washings was to remove sulphates and carbonates from the soil. Modification was first attempted using only acid washings. When initial results were obtained from cation exchange analysis, they indicated a presence of large amounts of salt in the pore water. Upon microscopic examination of the material retained on the No. 200 sieve gypsum particles

were observed; thus it was concluded that a large portion of the sulphates had not been removed. At this time, ammonium acetate was suggested as a good electrolyte which would aid in breaking down and dissolving gypsum particles which were not being completely removed by the hydrochloric acid. Cation exchange analysis on samples modified according to the new procedure gave the same results as did the earlier analysis with the exception of the sodium samples, which were now essentially free of pore-water salts. Personal consultation with Dr. Pawluk, Department of Soil Science, resulted in the feeling that sulphate and carbonates were probably removed but the high concentration of acetate ions used to modify the soil was exceeding the capacity of the alcohol to remove them in a limited number of washes since calcium and magnesium acetates are somewhat insoluble in ethanol. This will be discussed further in paragraph 4.5. The soil modified using ammonium acetate and hydrochloric acid was thus felt to be acceptable and this procedure is recommended for further studies involving soil modification. The soil was subsequently washed with an acetate solution of the desired cation. The procedure was based on previous work (Thomson, 1963; Locker, 1963).

4.5 The final adopted procedure for removing pore-water salts included the reduction of their concentration by washing with very dilute solutions (0.001 N) of the desired cation and then washing with chemically pure ethyl alcohol. Originally only ethyl alcohol was used in this salt-removing process. After cation exchange capacity results were obtained and a discussion with Dr. Pawluk concluded, it was decided

that the concentration of salts in the pore water had to be reduced before the ethyl alcohol washes would be effective. The reason for using the acetate salt in modifying the soil is that the acetate radical is soluble in ethyl alcohol. As was shown in this research, the concentration of pore-water salts should be reduced in order to remove pore-water salts in a limited number of ethyl alcohol washings. Dilute washes of the desired cation were used in place of distilled water in order that hydrogen dissociation, which has a concentration of 10^{-7} gm-atom/litre in pure water, would have a minor effect; that is, the hydrogen cation would not become adsorbed on the clay particle.

4.6 In summary, the recommended procedure for producing a salt-free homionic soil, at least for the soil used in this thesis, is as follows:

- (a) Wash air-dried soil with a 1.0 N solution of ammonium acetate in water. This aids in subsequent exchange by tending toward an ammonium soil and also aids in subsequent removal of sulphates.
- (b) Wash soil with 0.75 N hydrochloric acid. This removes carbonates and sulphates.
- (c) Wash with distilled water. This reduces the concentration of acid in the pore water.
- (d) Wash with 1.0 N aqueous solution of the salt of the desired cation. This brings about the homionic state,
- (e) Wash with 0.001 N solution of the desired cation. This reduces the concentration of salts in the pore water.
- (f) Wash with a mixture of 80% ethanol to 20% water to remove all salts from pore water.

Exchange Capacity Analysis

4.7 Cation exchange capacity analysis was performed by the Department of Soil Science using the Alberta Research Council Procedure. The only variation in the procedure was that approximately 5 grams of soil passing the No. 40 sieve were used in place of 20 grams of a coarser size. The total exchange capacity was performed by a Kjeldahl nitrogen determination on the leachate containing ammonium ions which were present in the adsorbed state after the first leaching procedure. The presence and amount of cations available is determined by titration and flame photometer tests on a solution leached from the original soil by ammonium acetate. A measure of salts in the pore water is obtained by summing the flame photometer and titration results and subtracting the cation exchange capacity results from nitrogen determinations. Results of exchange capacity analysis are shown in TABLE IV.1.

Moulding Triaxial Specimens

4.8 All the samples were formed by placing the soil surry in a one-dimensional consolidation apparatus and consolidating under vertical pressures in three stages up to a maximum pressure of approximately 1.3 kg/sq.cm.. The similarity of specimens within one soil type can be estimated by comparing initial void ratios. Results are listed on the Summary of Data sheet in APPENDIX C. Although the soil structure need not be identical at equal void ratios, the differences should be slight. Thus the moulding technique appears to be adequate for this program.

Drainage Aids

4.9 In all cases the samples were surrounded by slotted filter paper and contained five internal wool wicks in a symmetrical pattern. A bottom filter paper and porous stone were also employed. The longitudinal wool wicks are important in the CFS test as it is felt that equilibrium of pore pressures can be established more quickly. On inserting the wool wicks some disturbance in the sample occurs, but this is partly compensated for by the consolidation that the sample undergoes and also by the increase in strength due to the wick themselves. The longest drainage path subsequent to inserting the wicks is about 0.9 cm. compared to approximately 1.8 cm. with no wicks. The rate of consolidation is thus increased about six times (Schmertmann, 1962).

The Load Cell

4.10 To obtain a nearly constant rate of strain, a load cell was used to measure vertical loads. The strain energy stored in a proving ring prevents rapid and controllable curve hopping. This trouble was avoided by using an aluminum tube as a load cell which, due to its rigidity, reacts quickly to load changes, hence it is particularly suitable for this type of testing.

4.11 An aluminum tube, which had been preloaded some twelve times to eliminate hysteresis effects, was wired with a full bridge circuit to give a load cell with a maximum usable capacity of about 125 kg.. The sensitivity over the calibrated range was 0.03268 kg/micro-inch/inch strain. This sensitivity is about one-half that obtained by Dahlman (1965), but still sufficiently accurate to obtain the desired readings.

Backpressure and Pore Pressure Reaction

4.12 Once consolidation was complete and the sample was ready to be tested, a backpressure of 2 kg/sq.cm. was applied while simultaneously increasing the cell pressure also by 2 kg/sq.cm.. In this way backpressuring was accomplished with little or no disturbance to the specimen. The purpose of backpressuring was to dissolve any entrapped air, thus limiting its effect on pore pressure reaction, movement of water and volume change. Although the initial degree of saturation is high, the pore pressure reaction test generally results in low values unless backpressuring is used. The system was left under the backpressure for 24 hours in most cases to allow the system to come to equilibrium.

4.13 A pore pressure reaction test was performed by increasing the cell pressure by 1 kg/sq.cm., balancing the system and recording pore pressure build up. After performing one such pore pressure reaction test low pore pressure reaction results were obtained which were attributed to expansion of the tubing from the base of the sample, through the volume change indicator, up to the null indicator. At this time a valve was devised that could be attached to the base of the triaxial cell, such that on further pore pressure reaction tests the valve could be closed, thus eliminating the problem of expanding pore pressure lines. Pore pressure reaction was much higher following the above modification.

4.14 The volume of water entering the specimen under backpressure due to dissolving air bubbles was not measured. Since water is moving into and out of the sample, pore pressure lines expand and contract under varying pressures, small water losses occur at connections, air bubbles dissolve and come out of solution with varying pressures, water and air migrates through membranes, etc., one cannot expect final moisture contents to agree completely with calculations made using the results from the volume change indicator.

The CFS Test

4.15 Only a brief discussion of the CFS test will be presented here. For a more complete discussion see Dahlman (1965).

4.16 Since the rate of strain in a CFS test is related to the time in which equilibrium of the pore pressure can be established, it is logical that the rate be related to the consolidation curve. Schmertmann (1962) suggested that the CFS test be performed at a compression rate not greater than one percent axial strain per theoretical 100 percent consolidation time interval (t_{100}). This suggestion was followed with no difficulty in obtaining points on the deviator stress versus strain plot after about 0.5% strain had occurred. The effect of the rate of strain on CFS test results was examined by Schmertmann (Closure, 1962). He concluded that cohesion was similar in magnitude for all tests in spite of the greatly increased time for pore pressure distribution. The shear resistance mobilized at a given strain was found to increase with decreasing strain rate, with an ordinary experimental error of $\pm 2^\circ$.

The ratio between the fastest and slowest strain rate was about 5000 to 1 (4.34%/hr. to 0.00078%/hr.). Thus the strain rate is not critical but must be compatible with the permeability of the soil being tested.

4.17 All factors concerning strain are based on the original length of the sample as correction curves established by the Norwegian Geotechnical Institute (NGI Number 45, 1957) are based on original length.

4.18 The CFS test is based on a "hopping" technique resulting in two stress-strain curves, each for a pre-selected constant vertical effective stress. During the "hopping" procedure, the specimen is either expanding or contracting depending on which effective vertical stress one is moving towards. Hence, there is a change in void ratio which may result in structural change. Therefore, to limit the structural change the magnitude of the "hop" or effective stress change should be limited. However, limiting the change in effective stress reduces the accuracy with which shear parameters may be measured. A compromise suggested by Schmertmann and Osterberg (1960) limited effective stresses to 75 to 100% of the cell pressure used at the time of triaxial testing. This produced strength changes that could be interpreted with sufficient accuracy and yet involve only small void ratio changes (usually less than 1%). It was decided for this program, since the specimens were overconsolidated, that values of the vertical effective stress be 1.0 and 1.2 times the cell pressure. Choosing a value for the high effective stress curve greater than the cell pressure should result

in slightly less void ratio change during "hops" and perhaps less structural change. Tests using a high effective stress curve greater than the cell pressure were tried by Dahlman (1965), which did not reveal any significant discrepancies in results.

4.19 The physical significance of the CFS test is limited in field application to its similarity to repeated rebound and recompression cycles. However, as a research tool it appears to present a more comprehensive picture of the source and nature of shear strength of cohesive soils.

CHAPTER V

PRESENTATION OF RESULTS

Introduction

5.1 The purpose of this thesis was to investigate the effect of preconsolidation on the mobilized components of shear strength, cohesion and friction, with strain. The results of the CFS test are presented in this chapter, leaving the interpretation and discussion of the results to Chapter VI.

Computed Cohesion and Friction

5.2 The mobilized cohesion and angle of internal friction at any strain were computed using the equations presented in paragraph 2.3. The data necessary for the computations were obtained from the deviator stress versus strain plots which are in APPENDIX A. A typical set of complete data sheets and calculations are given in APPENDIX B. Computations were performed during the progress of the test at intervals of strain no larger than 1% to obtain points for the best fit curve that defines mobilized cohesion and angle of internal friction. The curves thus obtained are presented in FIGURES V.1 to V.6.

5.3 The curves presented in FIGURES V.1 to V.4 show the magnitude of the mobilized cohesion and angle of internal friction versus axial compressive strain for a given soil modification. The cohesion and

FIGURE V.1

COHESION VS AXIAL COMPRESSIVE STRAIN
FOR CALCIUM MODIFICATION

NOTE: - ALL SPECIMENS PRECONSOLIDATED TO 8 KG/SQ. CM.
- CELL TEST PRESSURE NOTED IN [] KG/SQ. CM.

COHESION, KG/SQ. CM.

AXIAL COMPRESSIVE STRAIN, %

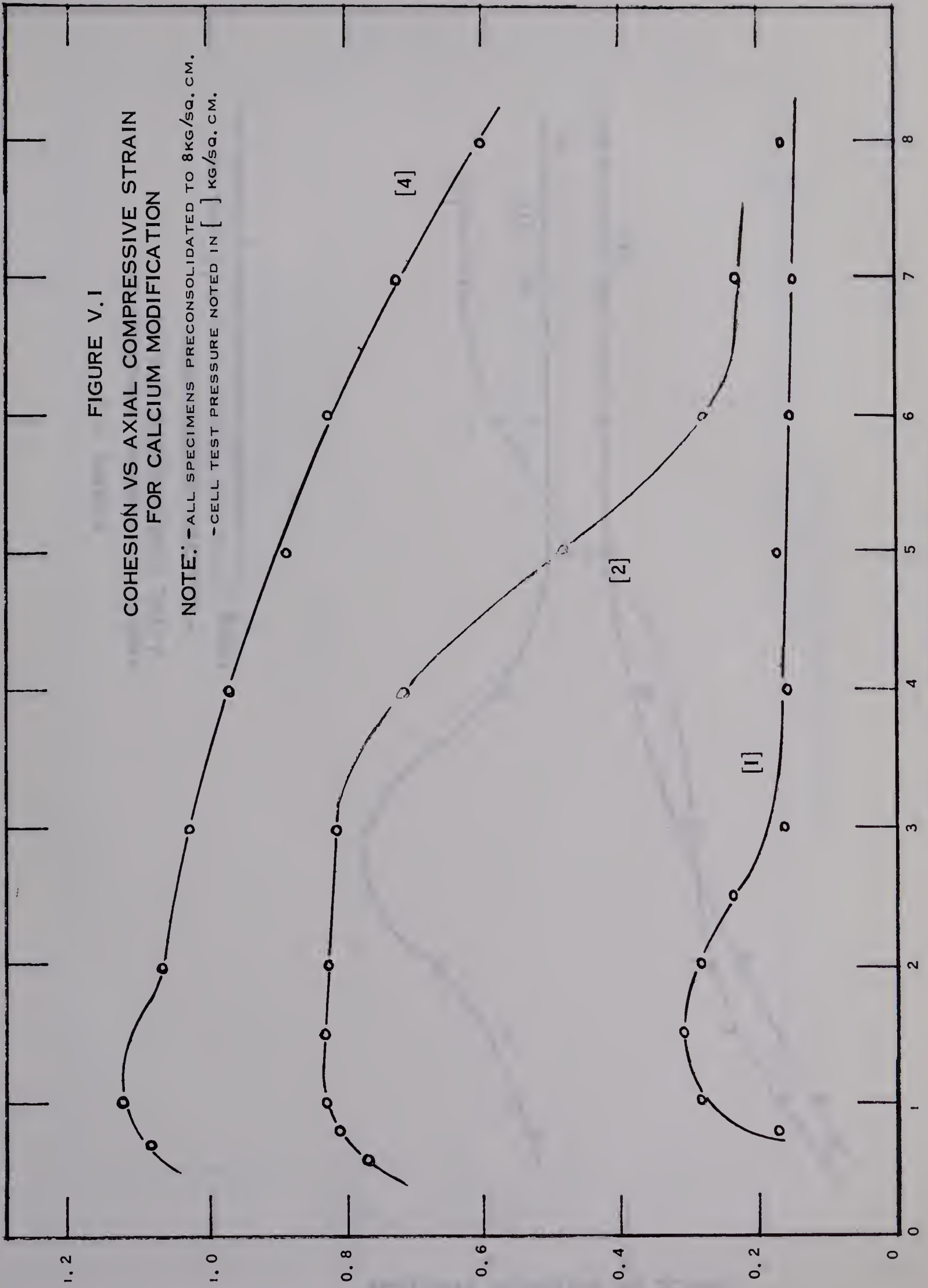
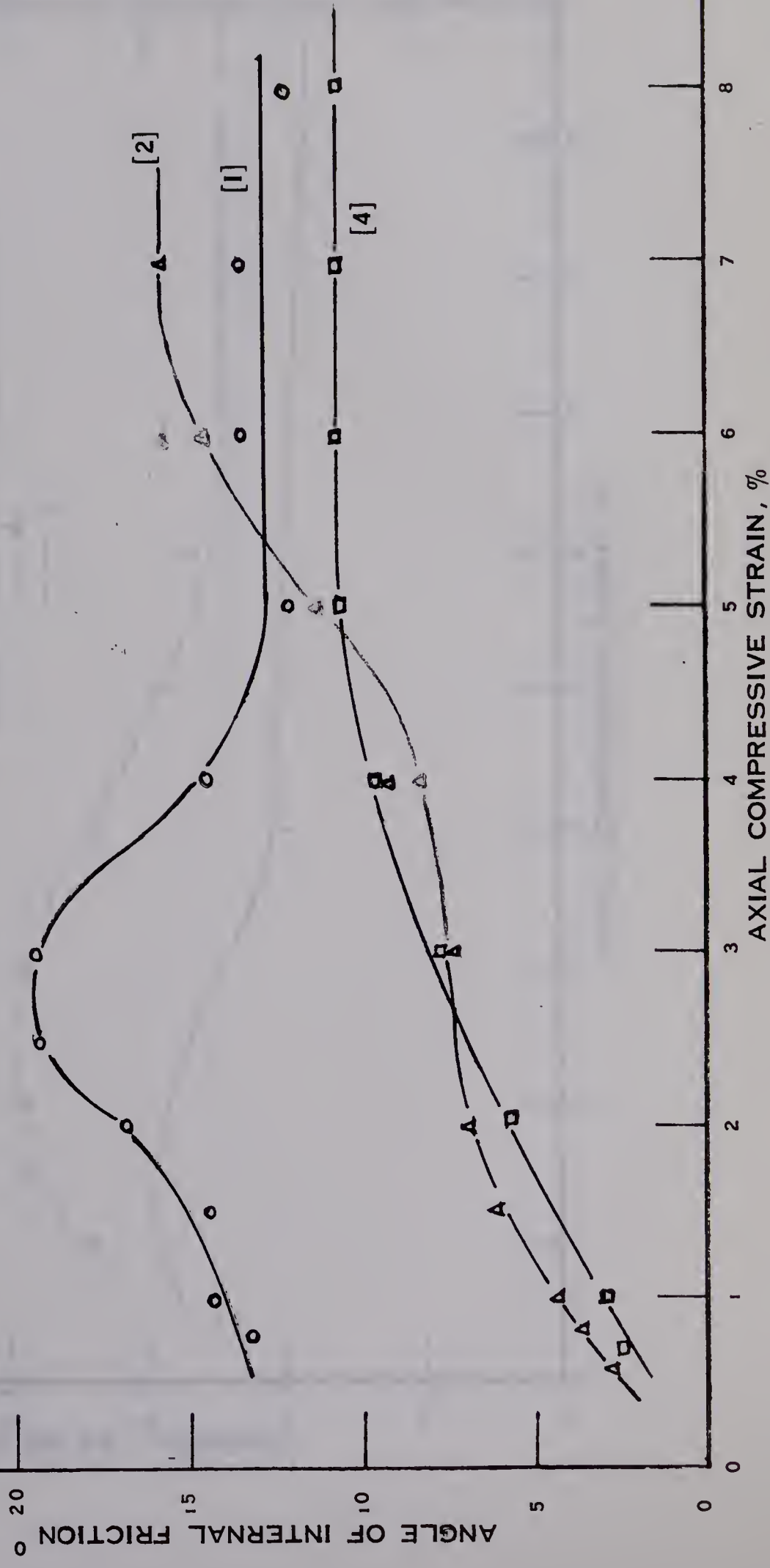
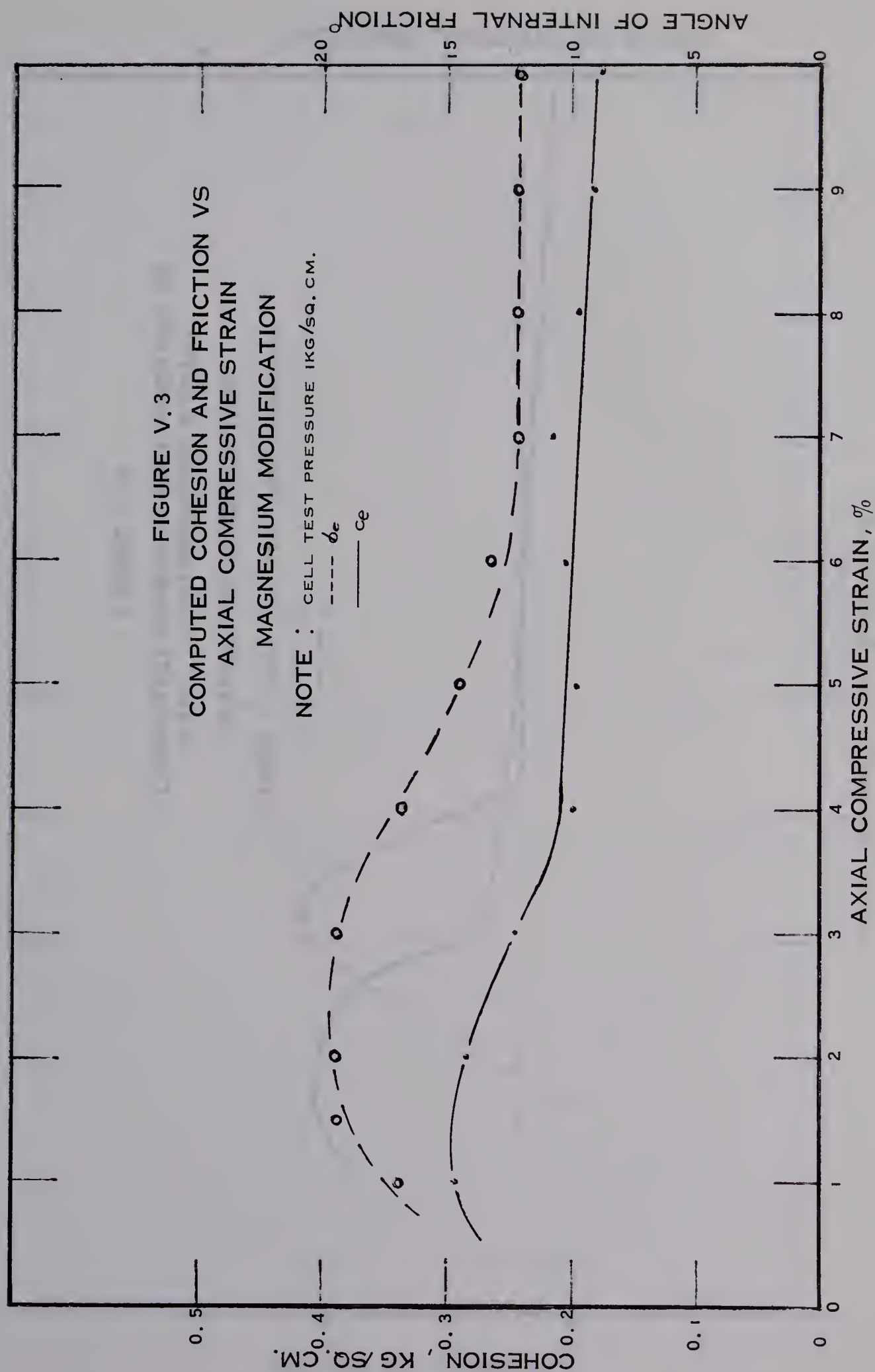
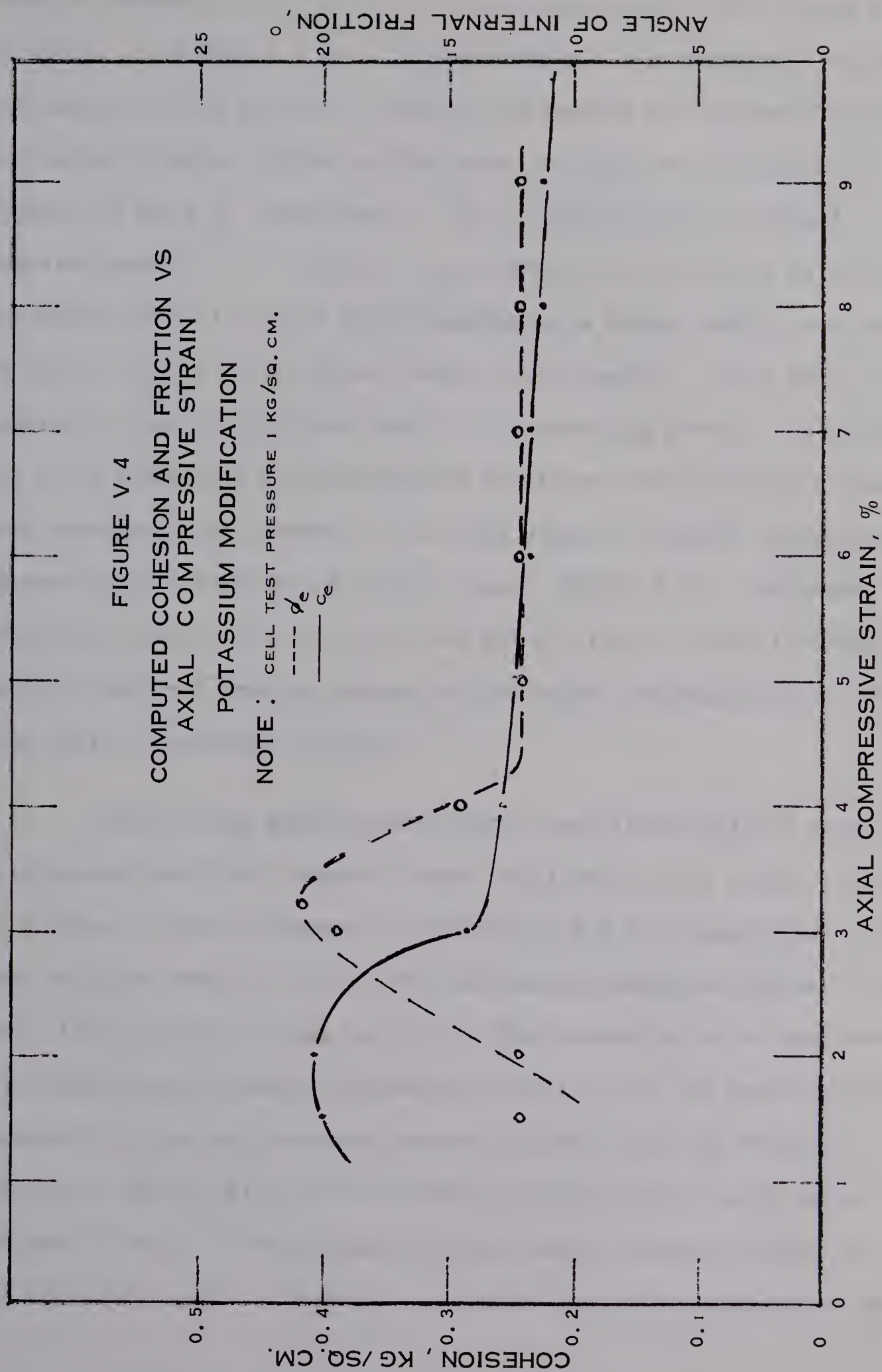


FIGURE V.2
 ANGLE OF INTERNAL FRICTION VS
 AXIAL COMPRESSIVE STRAIN
 - CALCIUM MODIFICATION

NOTE : - CELL TEST PRESSURE NOTED IN [] KG/SQ. CM.
 - ALL SPECIMENS PRECONSOLIDATED AT 8 KG/SQ. CM.

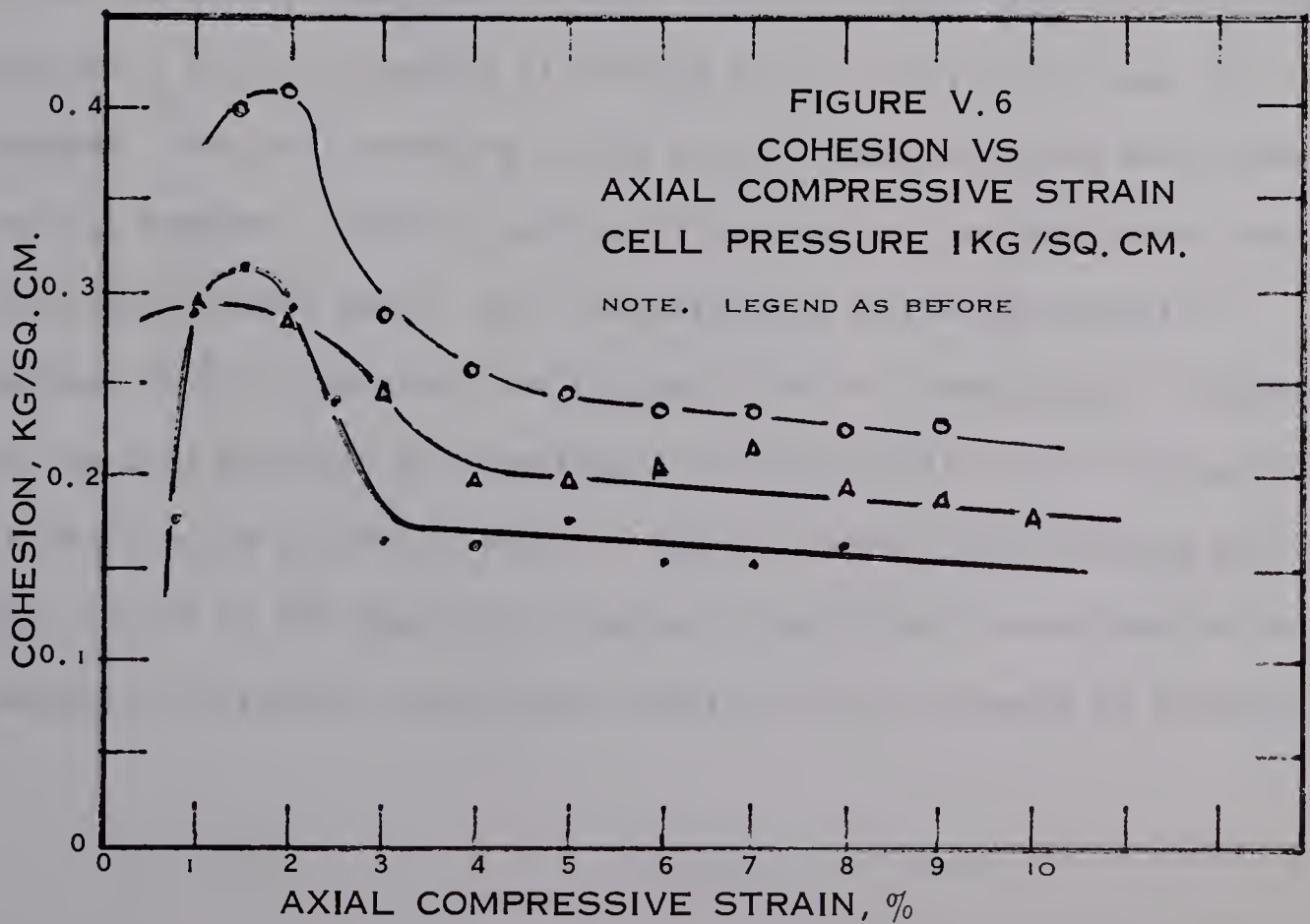
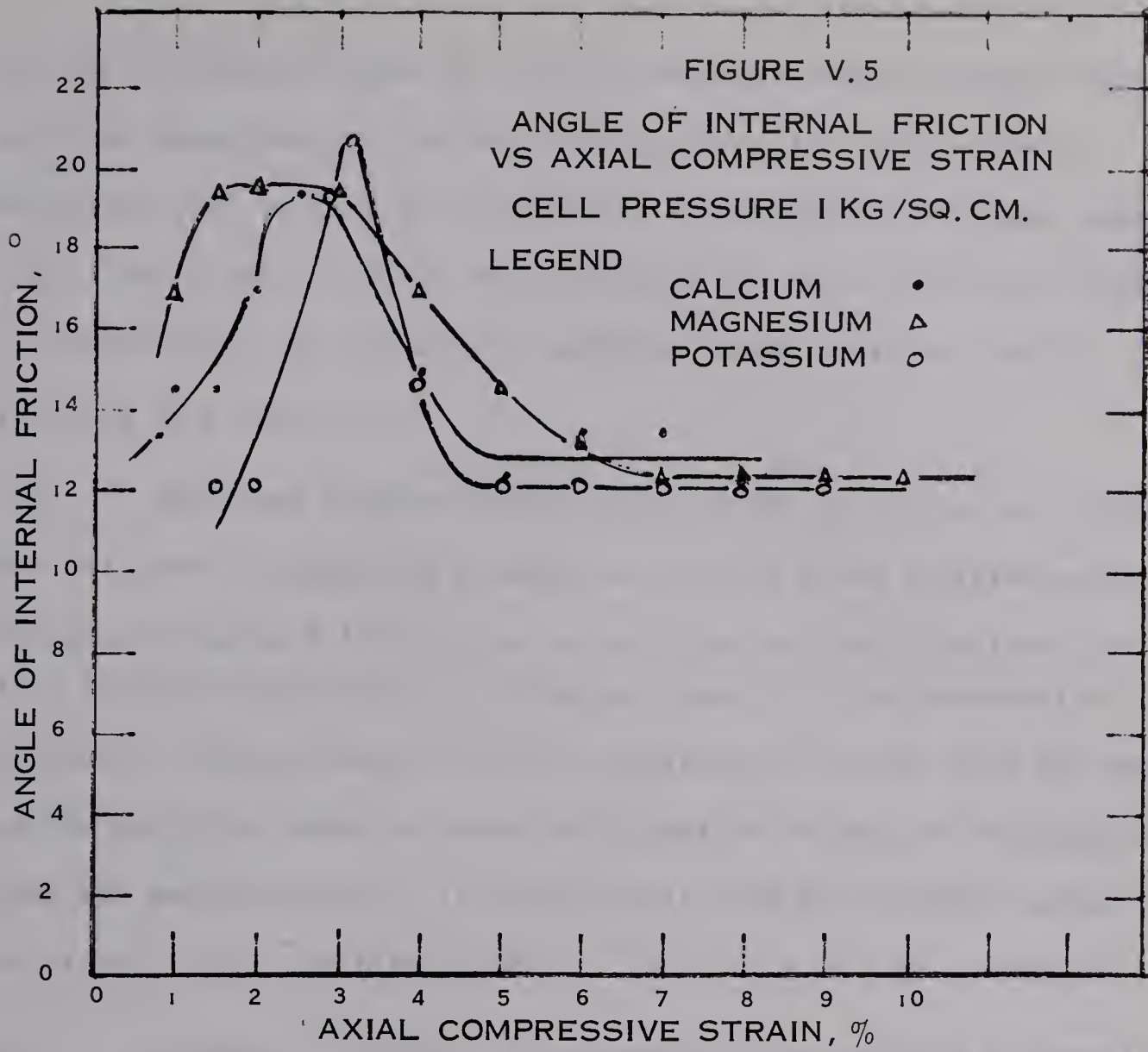






angle of internal friction for the three calcium-clay soils tested are presented in FIGURES V.1 and V.2 respectively. One potassium-clay and one magnesium-clay were also tested. The results of computed cohesion and angle of internal friction for these two soils are presented in FIGURE V.3 and V.4, respectively. All of the curves of mobilized cohesion peaked at low strains 1 to 2%, then dropped sharply to a value approximately half that of peak cohesion, at a strain roughly equivalent to that of little or no volume change of the specimen. From this point, mobilized cohesion decreased slowly with increasing strain. In FIGURE V.1 it is shown that the magnitude of mobilized peak cohesion increased with increasing cell pressure. The peak angle of internal friction also appears to be a function of stress history (FIGURE V.2). The higher the overconsolidation ratio the lower the axial strain at which the peak angle of internal friction occurs and the higher the magnitude of the peak angle of internal friction.

5.4 For the soil modifications tested, mobilized angle of internal friction and mobilized cohesion versus axial strain, for a cell pressure of 1 kg/sq.cm., are presented in FIGURES V.5 and V.6, respectively. The peak angle of internal friction for calcium and magnesium are both the same, 19.4° , and for potassium 20.8° . The limited amount of test data available, plus the small difference in magnitude of the peak angle of internal friction with various cations, suggests that the adsorbed cation (Ca, Mg and K) has little effect on the mobilized peak angle of internal friction. The residual friction angles (defined herein as the constant magnitude of friction noted at high axial compressive strains,



and not necessarily equal to residual strength values obtained from a direct shear test on the same material) are also very close in magnitude (12° to 13°) and all occur at approximately the same strain (5%). The strain at which the peak angle of internal friction occurs is approximately 3% for all the modified soils tested at a cell pressure of 1 kg/sq.cm..

5.5 Mobilized cohesion versus axial strain is plotted in FIGURE V.6. The cell test pressure was 1 kg/sq.cm. for all three modified soils. Cohesion peaks at a low strain, for all the soil modifications, and then decreases slowly with increasing strain. The potassium-clay exhibits a greater cohesion at all magnitudes of strain with the shape of the mobilized cohesion curve being similar to that of the calcium-clay and magnesium-clay. It would appear that the adsorbed cation has an effect on the mobilized cohesion, especially at higher strains.

5.6 To study the effect of preconsolidation, samples are required which are nearly identical in every respect, except for their stress history. Also, the method of testing must be nearly the same for all samples. The cell pressure at the time of testing should be the same for all samples. The only series of tests which included more than one overconsolidation ratio, was that performed on the calcium-clay. Dahlman (1965) concluded that the magnitude of cohesion was a function of the cell pressure or consolidation pressure with which the soil was in equilibrium prior to testing. However, none of the calcium specimens were tested at the same cell pressure, thus direct comparison was not possible. Normally consolidated calcium-clay was tested by Dahlman (1965)

at the same cell pressures as used for two specimens in this program, Thus, a comparison between normally consolidated samples and overconsolidated samples at the same cell pressure can be made. It must be remembered that the modification procedures used in the two programs were not identical, thus the modified soils may differ slightly. In FIGURE V.7, a plot of peak cohesion, from both overconsolidated and normally consolidated tests, versus overconsolidation ratio is given. Arrows are drawn between presumably identical samples tested at the same cell pressure to show the increase in peak cohesion due to overconsolidation. Although the actual numerical values are of little interest, there is a definite indication that overconsolidation increases peak cohesion for the soil modifications tested in this phase of the program. Since all shear strength tests were performed within a short period of time after rebound was complete, the possibility of a reduction of cohesion with time was not considered. Due to obvious time limitations, there were no tests run to determine if a cohesion loss occurs with time. Thus, it appears that peak cohesion is a function of the consolidation pressure with overconsolidation resulting in a hysteresis or stored energy effect which may or may not be lost with geologic time.

5.7 Since consolidation pressure does not appear to influence the magnitude of the angle of internal friction, preconsolidation effects can be plotted for all three calcium-clays. FIGURE V.8 relates the ratio of peak friction angle to residual friction angle, to the overconsolidation ratio. The ratio of peak friction angle to residual angle, referred to in this program as the "peaking effect", is shown to increase

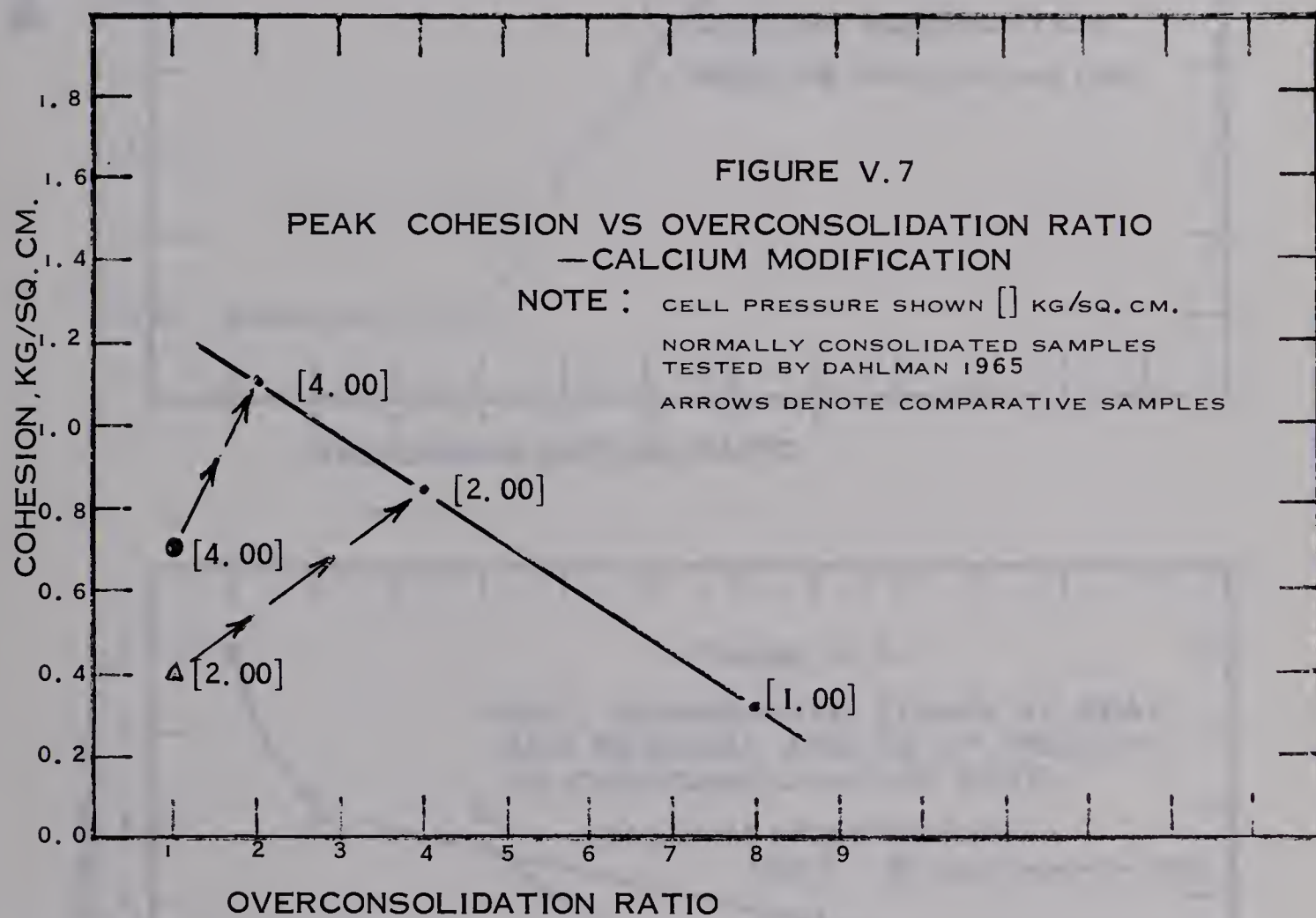
FIGURE V.7

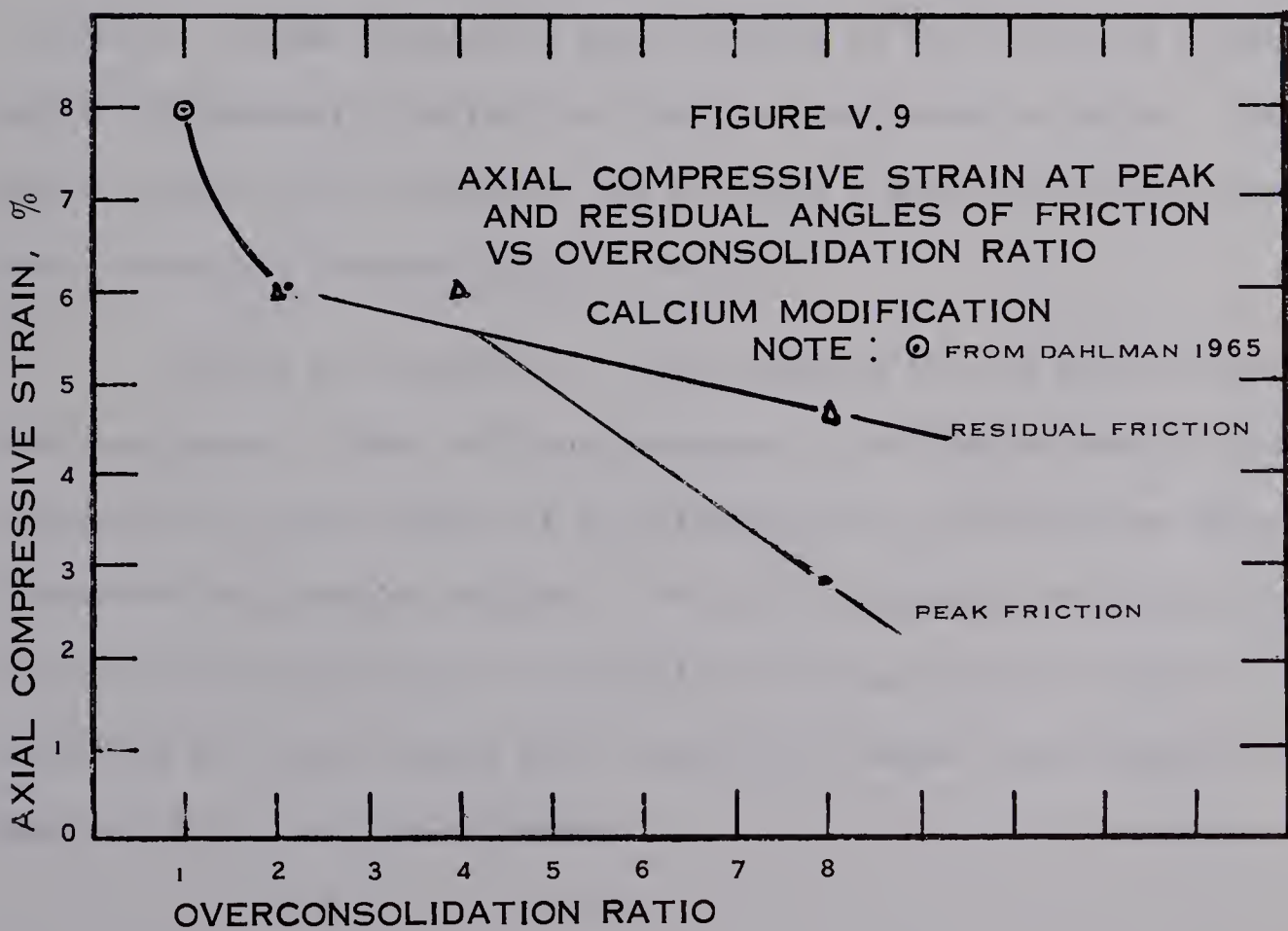
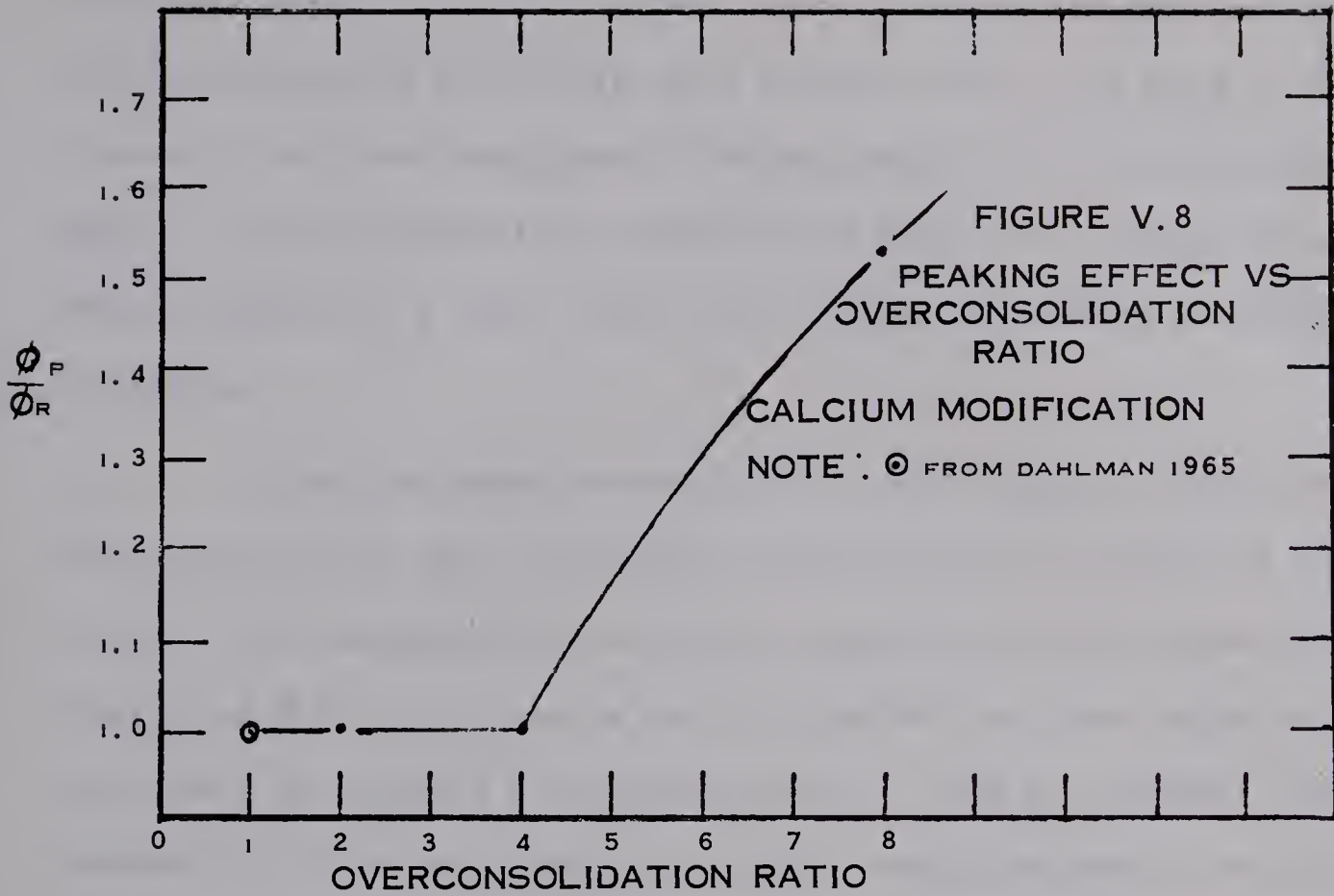
PEAK COHESION VS OVERCONSOLIDATION RATIO
—CALCIUM MODIFICATION

NOTE : CELL PRESSURE SHOWN [] KG/SQ. CM.

NORMALLY CONSOLIDATED SAMPLES
TESTED BY DAHLMAN 1965

ARROWS DENOTE COMPARATIVE SAMPLES





with overconsolidation ratios greater than four. In other words, for overconsolidation ratios less than four, the mobilized angle of internal friction increases with strain to a maximum value after which it remains constant. For overconsolidation ratios greater than four, the mobilized angle of internal friction increases with strain to a maximum value, then decreases to a lower value where it remains constant as strain increases.

5.8 It was mentioned previously that peak cohesion occurred at very low strains (1 to 2%), and varying overconsolidation ratios had little effect on the magnitude of the axial compressive strain at peak cohesion (paragraph 5.3). The same is not true for the mobilized angle of internal friction. In FIGURE V.9 the axial strain at peak and residual angles of internal friction are plotted for various overconsolidation ratios for the calcium modified soil. Although there is some scatter of results, a trend is evident indicating lower strains at both peak and residual angles of internal friction for high overconsolidation ratios. This would indicate that the sample was assuming a more brittle behaviour with increasing overconsolidation ratios.

5.9 FIGURE V.10 presents the Mohr Diagram for the three calcium-clay specimens. Since only one specimen of both magnesium-clay and potassium-clay was tested, it is not possible to obtain shear strength parameters by standard methods. The calcium-clay was analyzed by the Krey-Tiedemann analysis. The results are summarized in TABLE V.1, including CFS test results plus comparative values from other research work at the University of Alberta.

FIGURE V.10
MOHR DIAGRAM
CALCIUM MODIFICATION

KREY-TIEDEMANN ANALYSIS

$$\begin{aligned}\phi'_r &= 15.5^\circ \\ \phi'_c &= 5^\circ \\ \beta &= 20^\circ\end{aligned}$$

EFFECTIVE SHEAR STRESS, KG/SQ.CM.

EFFECTIVE NORMAL STRESS, KG/SQ.CM.

PRECONSOLIDATION
PRESSURE

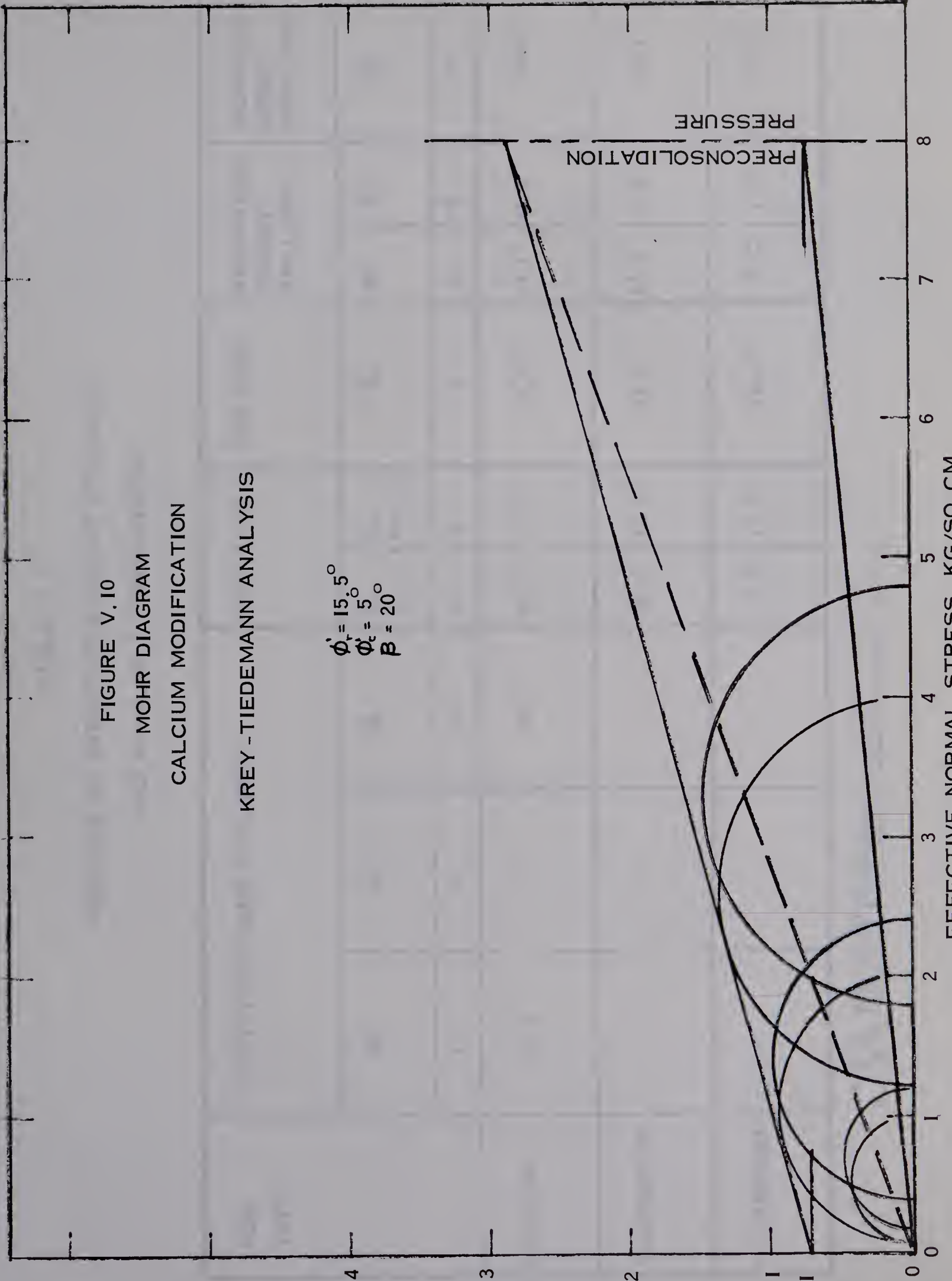
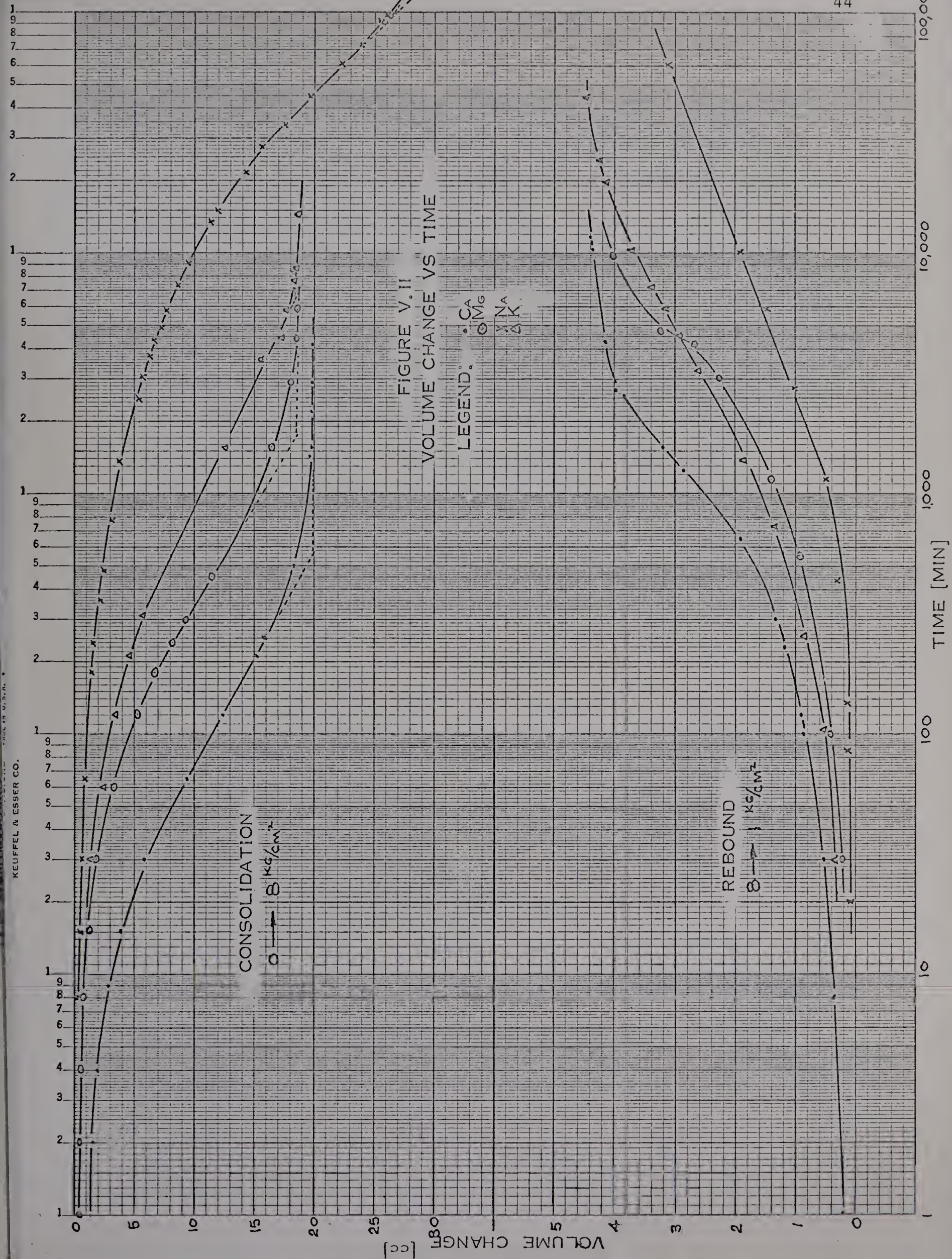


TABLE V.1
METHODS OF INTERPRETATION OF CFS DATA
AND AVAILABLE COMPARISONS

SOIL TYPE	KREY-TIEDEMANN ANALYSIS				CFS TEST		CFS TEST	KREY-TIED- EMANN ANALYSIS		MOHR-COU- LOMB ANALYSIS
	σ'_R	σ'_c	β		σ_E MAX.	σ_E RESID.	σ_E	σ'_R	σ'_c	β
	1	2	3		4	5	6	7	8	9
CALCIUM	15.5°	5°	20°		19.4°	12.7°	12.0°	11.1°	6.9°	21.0°
MAGNESIUM	--	--	--		19.4°	12.3°	13.6°	12.5°	6.5°	21.1°
POTASSIUM	--	--	--		20.8°	12.1°	18.6°	14.0°	7.0°	27.2°

1, 2, 3. SEE FIGURE V.10
4, 5. CFS TESTS [OVERCONSOLIDATED]
6, 7, 8, 9. FROM DAHLMAN, 1965 [NORMALLY CONSOLIDATED]



5.10 The consolidation and rebound curves for all the soil modifications are presented in FIGURE V.11. Each of these samples was pre-consolidated at 8 kg/sq.cm. and then allowed to rebound under a cell pressure of 1 kg/sq.cm.. The sodium-clay expelled a great deal more water in a much greater period of time than the other three samples. The initial moisture content of the sodium-clay was also much higher than for the other samples. The time at which the rate of rebound reduces noticeably is longer than the theoretical 100% consolidation time for all the samples. Extrapolation of the sodium rebound curve suggests that some 2.5 to 3 years would be required for equilibrium to be reached. Also the quantity of water being taken in is far less than that expelled during consolidation. It may also be noticed that the relative positions of the curves are the same at theoretical 100% consolidation and maximum rebound times.

Summary

5.11 The results of the CFS tests performed were presented in this chapter. Cohesion was shown to peak at low strains, almost independent of the overconsolidation ratio. The angle of internal friction was shown to peak at higher strains, with the strain at peak reducing as the overconsolidation ratio increased. The magnitude of the cohesion appeared to be a function of the cell pressure and independent of the type of adsorbed cation. A residual cohesion was not obtained for any of the soil modifications. The magnitude of the cohesion continued to decrease with increasing strain past that required for peak cohesion. The mobilized angle of internal friction did not appear to be a function

of cell pressure at the time of testing and only slightly dependent on the type of adsorbed cation. However, as the overconsolidation ratio increased, the magnitude of the peak angle of internal friction also increased. Both a peak and residual angle of internal friction were obtained for all the soil modifications tested at 1 kg/sq.cm. by triaxial testing.

5.12 A Krey-Tiedemann analysis was performed on the CFS test data and the results were compared with previous computations. The results are in reasonably close agreement but the lack of shear strength and accurate moisture content data limits the analysis of these results.

CHAPTER VI.

DISCUSSION OF RESULTS

Introduction

6.1 As was stated previously, paragraph 1.1, physical-chemical interpretation of shear strength results involves some controversial aspects. The microscopic nature of clay particles tends to hinder analysis because of the lack of feeling, on the part of engineers, for the interaction between fine grain particles.

6.2 The purpose of this thesis was to extend the results obtained by Dahlman (1965), who tested normally consolidated soils, to overconsolidated soils. The first portion of the discussion will attempt to relate the present work on overconsolidated soils to that previously presented on normally consolidated soils. The second portion of the discussion will attempt to show the effect of overconsolidation on the shear strength parameters. For purposes of comparison, results for normally consolidated soils have been obtained from research by Dahlman (1965).

6.3 The usual methods of representing shear strength parameters employ the Mohr Diagram which represents stress conditions within a mass. The Mohr failure envelope is the line representing the locus of points showing stress conditions of the failure plane. The Coulomb-Hvorslev failure criterion, which forms the basis for calculation of

cohesion and angle of internal friction at any strain during the CFS test, is simply the equation of a Mohr failure envelope. Schmertmann and Osterberg (1960) justified the use of this equation at any strain by the following statement:

"Since a Mohr's circle represents any two-dimensional stress condition of equilibrium, it can be used at any strain value before failure is reached with the same validity as at the failure conditions conventionally chosen. The envelope formed by points of tangency to the Mohr circles representing intergranular stresses at a point for a given value of strain can be used to find the cohesion and friction at that strain. If the Mohr circles for different intergranular stresses are all found at the same soil structure, the cohesion and angle of internal friction are obtained."

6.4 The condition of "... without significant change in soil structure" (paragraph 2.2), imposed by the use of one sample to obtain two stress strain curves and thus the shear strength parameters, is a very important statement. The void ratio does change slightly during curve "hopping", thus there must be some change in soil structure. However, it is felt that the change does not effect the results of the CFS test to such a degree that trends shown are invalid (Schmertmann, 1962; Dahlman 1965). Two-specimen CFS tests were performed by Schmertmann (1962) in which no curve "hopping" was done and the results were compared to one-specimen tests performed in a manner similar to the procedure used in this program. The results obtained indicated a close agreement between the two methods of test.

6.5 The effect of the water hull arrangement can be roughly estimated by examining the consolidation and rebound curves as well as the moisture content data. It was found that the sodium-clay had the

highest initial moisture content and also expelled the greatest quantity of water in the longest period of time. This is believed to result from thick, weakly bound water hulls. The thick water hulls would result in high initial void ratios and thus high initial moisture contents. The presence of thick water hulls would also impede the movement of water by physically rendering many pore passages discontinuous, thus resulting in long consolidation and rebound times. The nature of the calcium and magnesium water hulls do not vary appreciably in thickness, thus similar consolidation and rebound times were expected. The slightly lower times for the calcium-clay may have resulted from the diffuse layer having an abrupt boundary rather than a gradual transition from the diffuse to free water zones as is present in the magnesium-clay. The consolidation and rebound times for the potassium-clay were expected to fall between the extremes represented by the calcium-clay and the sodium-clay. This was found to be the case. The potassium cation, because of its smaller radius, is able to be positioned more closely to the clay particle or possibly even be located in the clay lattice. The tight bond thus formed results in a thin oriented layer with a well defined boundary.

6.6 The clay particles, along with the adsorbed cations, hydrate in the presence of water, thus a layer of tightly bound water forms around the soil particle with a thickness approximately equal to that of the hydrated radius of the adsorbed cation. Extending beyond this is a layer of water of variable thickness, orientated to a greater or lesser degree. At a moisture content of approximately the liquid limit, the interaction of the adsorbed water layers should be limited.

As consolidation occurs, more and more of these adsorbed water layers come into contact with each other. For isotropic consolidation, as occurs in normal triaxial testing, the overall average shearing stress is zero, but for any small area local shearing stresses are set up. Thus reorientation of particles and new electrical equilibriums occur to counteract the imposed consolidation pressure. When the sample is rebounded a new equilibrium is established but a residual effect will probably occur depending on the strength of the bonds developed and the time involved. The bonds may range from minor van der Waals to actual chemical bonds. The time allowed for the build up of bonds and the adjustment to new conditions will likely effect the strength and other characteristics of the bond. Time effects are thought to be small in normal laboratory testing periods but are no doubt important in geologic time. No tests were performed to determine the effect of time on mobilized cohesion or friction.

6.7 For the purpose of this project, cohesion may be thought of as resulting from intrinsic residual prestress due to particle reorientation and simultaneous electrostatic force adjustments occurring as a result of changes in stress.

6.8 The author's concept of friction is a combination of interaction and interference among particles. The interaction of particles is believed to be the result of electrical forces which depend on the applied normal stress. Interference of particles, which may be brought to mind by cohesionless materials, involves energy requirements to re-orientate particles and to cause dilatancy.

Mobilized Cohesion

6.9 The calcium-clay was the only soil type for which specimens were tested at different cell pressures. For the calcium-clay, as for all other soil types, the specimens were preconsolidated at 8 kg/sq.cm. and then allowed to rebound. The cell pressures chosen for the calcium-clay were 1, 2 and 4 kg/sq.cm.. Thus all the samples should be nearly identical except for stress history. The results show that as the cell pressure increases, peak cohesion increases. The same conclusion was drawn from research by Dahlman (1965). A plot of peak cohesion versus cell pressure at the time of test shows the dependency of cohesion on the moisture content or void ratio, which in turn is dependent on consolidation pressure and independent of other system characteristics, such as the adsorbed cation.

6.10 Since the specimens were originally thought to be pressure dependent, the research program allowed for the testing of different soil modifications at the same cell pressure. The results of mobilized cohesion versus strain for calcium-clay, magnesium-clay and potassium-clay, show curves of similar shapes but separate loci depending on the adsorbed cation. It is felt, however, that the separation shown should not be attributed solely to the adsorbed cation but may contain errors inherent in the test technique. It was shown by Dahlman (1965) that the range of test error for mobilized cohesion was approximately 0.1 kg/sq.cm. which exceeds the magnitude of separation of the above curves (approximately 0.04 kg/sq.cm.).

6.11 The peak cohesion was reached at low strains for all samples (1 to 2%) and then cohesion decreased with increasing strain. This was also similar to results obtained by Dahlman (1965) on normally consolidated soils with a similar modification procedure. The only difference was a definite peak in cohesion being noted in overconsolidated specimens with overconsolidation ratios of four or greater. This peak appears to be manifested by preconsolidation, again indicating that cohesion is pressure dependent.

Mobilized Friction

6.12 Previous work by Dahlman (1965) indicated the angle of internal friction to be independent of consolidation pressure and related to the type of adsorbed cation for normally consolidated soils. The most obvious difference in mobilized friction occurred within the sodium modification. The sodium-clay was not tested in the overconsolidated series due to the time element and there are no test results which definitely indicate a dependence of friction on the adsorbed cation for the soils tested. It is felt that the grain size distribution, which probably was almost uniform for all samples, and type of clay particles present have a pronounced effect on the mobilized friction. The hydrated radii of the various cations employed in the soil modification does not vary appreciably. The hydrated radius of the potassium cation is slightly smaller and thus may form thinner water hulls or the cation may be taken into the clay lattice directly. However, the valence of unity may result in the electrostatic forces being weaker than those for the bivalent cations adsorbed in the magnesium-clay and calcium-clay.

Effect of Overconsolidation

6.13 The effect of overconsolidation on mobilized cohesion is to increase the magnitude of the peak cohesion over that which would be obtained from a normally consolidated sample of the same soil modification tested at the same cell pressure. The peak cohesion still occurs at a low value of strain, 1 to 2%. The peak cohesion decreases rapidly with increasing strain for overconsolidation ratios equal to or greater than four, until a value is reached which is similar in magnitude to the cohesion obtained from a normally consolidated sample at the same strain. From this strain on, cohesion reduces slowly with increasing strain. Since the magnitude of cohesion appears to be dependent on the confining pressure, comparisons can only be made between similar samples tested at the same cell pressure.

6.14 Recalling the concept of cohesion as stated previously, the above effects on cohesion can be explained. It was postulated that the interaction of double layers, orientation of particles and resulting equilibrium of electrical forces was dependent on the consolidation pressure. Thus if a sample is preconsolidated under a certain pressure, equilibrium can be established in time between attractive and repulsive forces. Now, if the confining pressure is reduced in magnitude significantly, this equilibrium will be disrupted and a new equilibrium established with time. The amount of readjustment that occurs is dependent on the nature of the bond that was established under the preconsolidation pressure. For the soils tested in this program, the magnitude of peak cohesion lies approximately midway between values that would be

obtained for normally consolidated samples tested at the preconsolidation pressure and the final cell pressure after rebound. Thus some loss in peak cohesion by reducing the confining pressure occurs, but the resulting cohesion is still higher than that obtained for a normally consolidated soil tested at the same cell pressure. The larger peak cohesion obtained for the overconsolidated sample as compared to the normally consolidated sample at the same cell pressure can be attributed to stronger structure. This stronger structure results from increased double layer interaction and particle reorientation with different electrical forces acting as a result of preconsolidation. Upon rebound, for the soils tested, structural change did occur but much of the energy of preconsolidation remained in the system. This energy manifests itself in greater structural bonding which results in a higher peak cohesion for the overconsolidated soil.

6.15 The effect of straining on the stronger bonds results in a further reorientation of particles. This reorientation of particles and adjustment of electrical forces tends to destroy the overconsolidation effect by disturbing the structure. This is measured on a macroscopic basis by an increase in void ratio with increasing strain. After considerable straining has occurred (depending on the overconsolidation ratio) the cohesion of the overconsolidated sample is similar to that of a normally consolidated sample at the same magnitude of strain. It is from this strain on that cohesion decreases much more slowly. This slow decrease in mobilized cohesion may be attributed to an increase in repulsive forces due to the reorientation of particles on the failure

plane. At this large strain the rate of volume change which contributed to the large initial loss in cohesion is almost zero, but a parallel orientation on the failure plane would result in similarly charged faces of clay particles being adjacent to one another. This would cause increased repulsive forces and thus a reduction in electrical bonding. It is also possible that the moisture content on the failure plane increases due to increased orientation in this zone at high strains. This would disturb the equilibrium and result in a lower mobilized cohesion.

6.16 The effect of overconsolidation on the mobilized angle of internal friction is to cause a peak and residual friction to be obtained, for the soils tested, at an overconsolidation ratio of 8. The axial strain at which the peak angle of internal friction occurs also decreases with increasing overconsolidation ratios. At strains greater than that at peak friction, for the soils tested and an overconsolidation ratio of 8, the magnitude of friction decreased to a residual value which it maintained for the duration of the triaxial test. For the calcium soils tested at overconsolidation ratios of 2 and 4, and for normally consolidated samples tested by Dahlman (1965), the friction angle increased to the peak value which it maintained for increased strain. The magnitude of the peak angle of internal friction increased with increasing overconsolidation ratios, but the residual angle of friction appeared to be independent of the overconsolidation ratio and cell pressure at the time of testing. Also the adsorbed cation appeared to have no measurable effect on the magnitude of the

peak or residual friction angle as obtained from the triaxial test. The strain at which peak and residual friction occurred also did not vary appreciably with different adsorbed cations.

6.17 Recalling the concept presented in paragraph 6.5 of friction resulting from interaction and interference of particles, the above effects of overconsolidation can be explained.

6.18 Overconsolidation results, for the soils tested, in a denser structure with stronger and perhaps more brittle bonds between the particles than would be present for a normally consolidated soil of the same type and at the same cell pressure. This increased density of the structure can be measured on a macroscopic basis by the void ratio. The orientation of particles was not examined, thus any effects this may introduce are of unknown magnitude. The electrical forces which would arise due to imposed normal stresses probably result in a new equilibrium of bonding being established between particles. Thus there will be varying bonding strengths for various overconsolidation ratios. Samples tested in this program which are overconsolidated are actually in a state of pseudo-equilibrium. That is to say, that upon preconsolidation energy is put into the system which upon rebound is not completely recoverable in the short period of time allowed for rebound. Thus the particles and bonding which maintains the structure are not in full equilibrium with the new cell pressure after rebound. Thus the more highly overconsolidated the sample, the more unstable the structure is after rebound. For samples rebounded from a common

preconsolidation pressure to different pressures, such as was done in the calcium-clay series, the sample at the lowest pressure may have rebounded slightly more than the other samples, but the structure is proportionately more highly unstable at the lower cell pressure. Thus, the peak friction occurs at a lower strain. The magnitude of the peak angle of internal friction appears to increase with increasing overconsolidation ratios. This effect was observed in the calcium-clay series. If the Mohr Diagram for the calcium-clay series is examined, it can be seen that the Mohr rupture line curves downward sharply towards zero shear strength for a low normal stress. This has been observed in other research and was explained by Tschebotarioff (1952) in the following way:

"...the shear strength tends toward zero at zero normal pressure, because, as shown by the void ratio pressure diagram the expansion of a fully submerged specimen under decreasing pressure increases rapidly when low normal pressures are reached."

Thus the Mohr envelope may be steep at low normal pressures but this results from a rapid loss in shear strength, not the result of a large angle of internal friction. This effect may have predominated in all three soil modifications tested at 1 kg/sq.cm., thus resulting in high friction angles being obtained as compared to results from normally consolidated samples tested by Dahlman (1965). But, the magnitude of peak friction should be larger for overconsolidated soils because the void ratio at a specific normal stress is lower than that associated with a similar normally consolidated soil.

Cohesion and Friction

6.19 The separation of cohesion and friction has been relied upon for many years for the foundation of shear strength theories. This originates from the concept of pure frictional and pure cohesive materials. The CFS test appears to be a useful research technique for providing two separate components of shear strength. Thus the mechanisms contributing to the shear strength are more easily understood.

6.20 The apparent dependency of both friction and cohesion on electrostatic forces make it very difficult to state which component is responsible or predominating in developing the observed shear strength. Thus the question arises concerning the validity of mathematically separating shear strength into two components by the CFS test. For research work, separation of shear strength into various components may be of value in studying the basic variables.

Summary

6.21 This chapter attempted to analyze and explain by physical-chemical phenomenon the results presented in Chapter V.

6.22 The use of the Mohr Diagram in determination of the shear strength parameters was explained by using the original ideas presented by Schmertmann. The effect of the "hopping" procedure on the shear strength parameters was discussed briefly.

6.23 The laboratory procedure of sample preparation was discussed in terms of strength build up and the effect of rebound on bonding was mentioned.

6.24 The concepts of cohesion and friction were presented and the results were discussed with these concepts in mind. It was concluded that cohesion was dependent on the stress history. For the higher overconsolidation ratios, a pronounced peak in the mobilized cohesion was noted and attributed to the preconsolidation effect resulting in stronger inter-particle bonds. Mobilized friction was found to be independent of the adsorbed cation and cell pressure at the time of testing. It was felt that the grain size distribution and the type of clay particles present played an important role in manifesting the observed mobilized friction. The effect of overconsolidation was to lower the strain at which peak friction occurred and to increase the magnitude as compared to normally consolidated samples because of the lower void ratio and stronger electrostatic forces present. The magnitudes of the friction angles obtained for the three soil types at the same cell pressure are in doubt because the shear strength was decreasing rapidly at low normal pressures and the strengths obtained are not believed to be on the Krey-Tiedemann rebound line.

6.25 The practical value of separating shear strength components was doubted because of dependency of both components on electrostatic forces and thus being so closely related. However, as a research tool separating the components can lead to a clearer understanding of shear strength.

CHAPTER VII

CONCLUSIONS

7.1 The following conclusions are drawn from the research presented in this thesis which consisted of a testing program on calcium-clay, magnesium-clay, and potassium-clay. The CFS test procedure was employed to test the remoulded, overconsolidated, homionic, highly plastic lacustrine clay.

7.2 Cohesion as defined by the CFS test was shown to be a function of stress history for the soils tested. Both the present confining pressure and the preconsolidation pressure influence the magnitude of the peak cohesion. The adsorbed cation appeared to have some influence on mobilized cohesion at higher strains but the variations noted were within the range of normal test error, thus no definite trend can be presented. The percent axial strain at which peak cohesion occurred, approximately 1 to 2% for all samples, did not appear to be effected by system characteristics or stress history. The higher the overconsolidation ratio the lower the axial strain at which the effect of preconsolidation is lost. This results from the structure being more highly unstable or the interparticle bonds more brittle. From this point on cohesion was found to decrease slowly with increasing strain. No residual value of cohesion was found.

7.3 Mobilized friction as defined by the CFS test was found to be independent of the cell pressure at the time of test and adsorbed cation for the soils tested. Preconsolidation increased the magnitude of friction because the void ratio was lower for a specific cell pressure than for a normally consolidated soil at the same cell pressure. The strain at which peak friction occurred decreased with increasing overconsolidation ratios. The magnitude of mobilized friction was found to increase with increasing overconsolidation ratios, for a specific soil type, but it is believed this represents a rapid loss in shear strength at low confining pressures rather than an increase in mobilized friction. A residual friction angle was obtained for all samples tested at 1 kg/sq.cm. which corresponded closely to peak values obtained from normally consolidated samples. It was felt that the grain size distribution and type of clay particles present play an important role in manifesting mobilized friction.

7.4 The practical value of separating shear strength components is doubted because of both components depending on electrostatic forces and thus being so closely related. However, as a research tool separating the components can lead to a clearer understanding of shear strength.

CHAPTER VIII

RECOMMENDATIONS

8.1 The following recommendations arise from the test procedure and analysis of the results.

8.2 The test procedure finally adopted did result in a homionic, salt free soil. However, this procedure is time consuming and tedious. If it is possible, modified soil should be prepared by technicians prior to further research or the soil should be bought from a manufacturer.

8.3 In order to know moisture content data accurately during testing, a more accurate system than burette readings should be employed. A closed system would be far superior. Also, cell water lines connecting volume change indicators to the samples should be as short as possible and sufficiently rigid to resist expansion and contraction during pressure changes.

8.4 Soils of low permeability should be avoided since the time for equilibrium becomes excessive and there is difficulty in establishing equilibrium accurately during testing. Also, the duration of the test becomes long and minor leaks, evaporation, etc. complicate the procedure.

8.5 If measurements were fed into a continuous recording device this would aid one in establishing deviator stress curves more quickly and accurately.

8.6 Further research should include samples preconsolidated at different pressures and then rebounded to one specific test pressure. At least one normally consolidated sample should be included in the test series. This is suggested since most comparisons are made at the cell pressure at the time of test.

8.7 Investigations of the effects of various pore fluids, both polar and non-polar, may reveal interesting modifications to mobilized cohesion and friction. In this way, it may be possible to compare samples in which the only variable is the adsorbed water hull. Thus friction and cohesion for similar samples with and without water hulls may be obtained and possibly explained on the basis of the adsorbed water layer.

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APPENDIX A

DEVIATOR STRESS VS STRAIN DIAGRAMS

FIGURE A.1

DEVIATOR STRESS VS AXIAL COMP. STRAIN
CALCIUM MODIFICATION

NOTE: - CONSTANT γ_i SHOWN ON CURVES
- CELL TEST PRESSURE () kg/cm²

4.80

4.00

(4.00)

(2.00)

(1.00)

DEVIATOR STRESS (τ_c / cm²)

AXIAL COMPRESSIVE STRAIN %

A.

FIGURE A.2

DEVIATOR STRESS VS AXIAL COMPRESSIVE STRAIN

POTASSIUM MODIFICATION # 7

NOTE: - CONSTANT γ , SHOWN ON CURVES (KG/CM^2)

- CELL TEST PRESSURE ($1.0 \text{ KG}/\text{CM}^2$)

1.20

1.00

(1.00)

AXIAL COMPRESSIVE STRAIN %

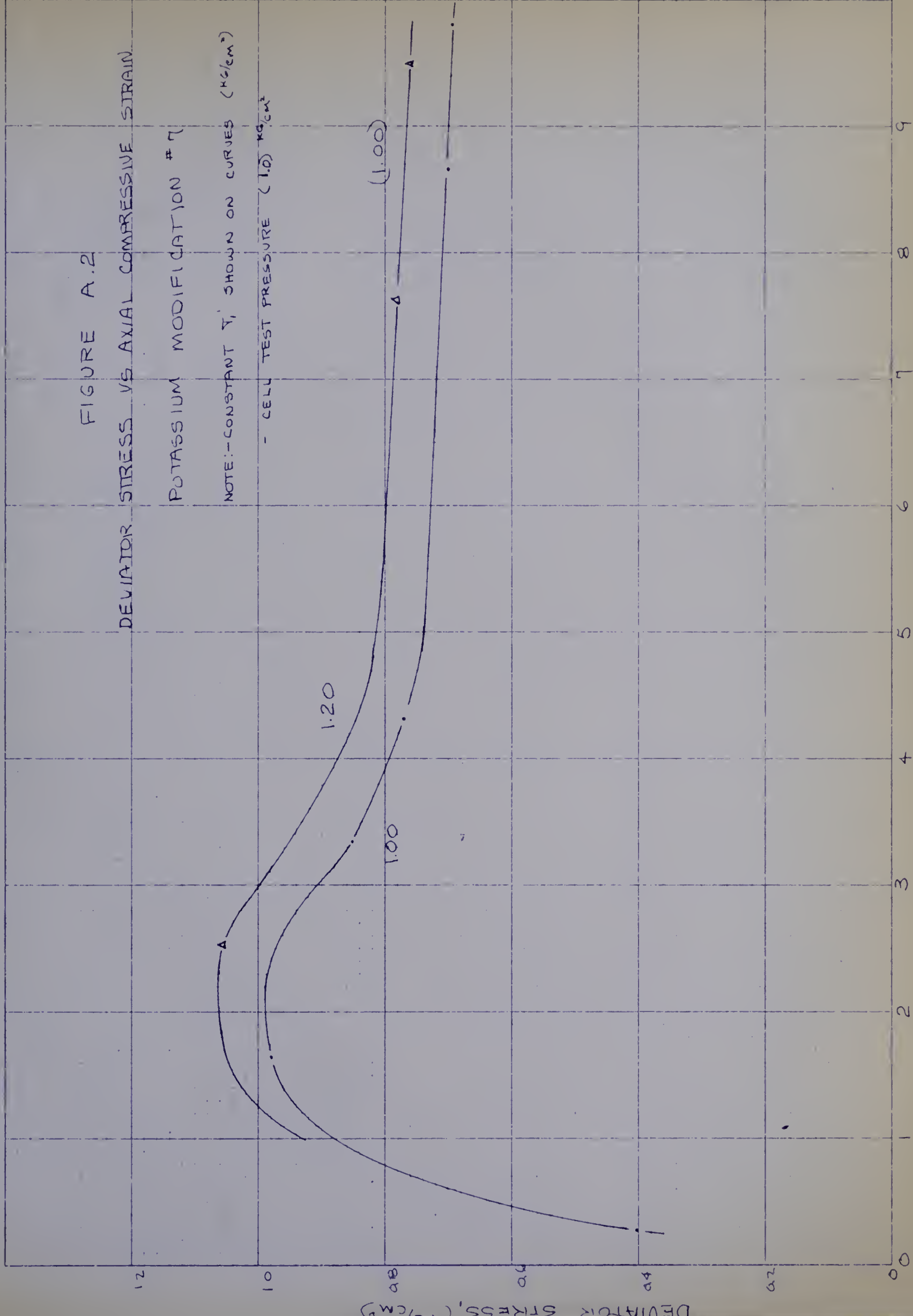


FIGURE A.3

DEVIATOR STRESS VS AXIAL COMPRESSIVE STRAIN

MAGNESIUM MODIFICATION # 10

NOTE: - CONSTANT γ' SHOWN ON CURVES, KG/CM^2

- CELL TEST PRESSURE (1.0) KG/CM^2

DEVIATOR STRESS, KG/CM^2

AXIAL COMPRESSIVE STRAIN %

A.3

12

10

8

6

4

2

0

2

3

4

5

6

7

8

9

1.20

1.00

(1.00)

APPENDIX B

TYPICAL DATA SHEETS AND SAMPLE CALCULATIONS

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

TRIAxIAL COMPRESSION TEST ON COHESIVE SOIL

Sample Description REMOULDEDLAKE EDMONTON CLAY- CALCIUM MODIFIEDInitial Weight 142.57 gmsFINAL WEIGHT 128.62 gms.Length, mm 1. 80.02 2. 80.12Diam. mm Top 1. 36.02 2. 35.88Centre 1. 36.16 2. 36.00Bottom 1. 36.00 2. 36.00Original Volume 81.58 cc

CONSOLIDATION DATA

8 KG/CM² (1 STEP)

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
FEB 17/67			
15:55	0.00	24.98	0.00
	6 ^s	24.75	0.23
	15 ^s	24.70	0.28
	30 ^s	24.63	0.35
	1 ^m	24.55	0.43
	2 ^m	24.40	0.58
	4 ^m	24.12	0.86
	8 ^m	23.64	1.34
	15 ^m	23.05	1.93
	30 ^m	21.55	3.43
20:10	255 ^m	13.60	11.38
FEB 18			
11:30	1175	8.50	16.48
11:30	BURETTE RESET TO 19.00		
13:45	1310	18.57	16.91
	AIR BUBBLE REMOVED - ADD 0.10		
FEB 19			
0:30	1955	17.95	17.63
12:15	2660	17.72	17.86
FEB 20			
8:55	3900	17.78	17.80

Project THESIS

Hole No. _____

Depth _____ sample #5 - Ca⁺⁺Engineer G. GILCHRIST

Technician _____

Date of Set-up FEB 17/67Test Lateral Pressure 2 KG/SQ CM.PRECONSOLIDATED AT 8 KG/CM²Aver. 80.07 Area Top A_T 1014.5 sq.mmAver. 35.95 Area Centre A_C 1021.9 sq.mmAver. 36.08 Area Bottom A_B 1017.4 sq.mmAver. 36.00 Average X-Sect Area

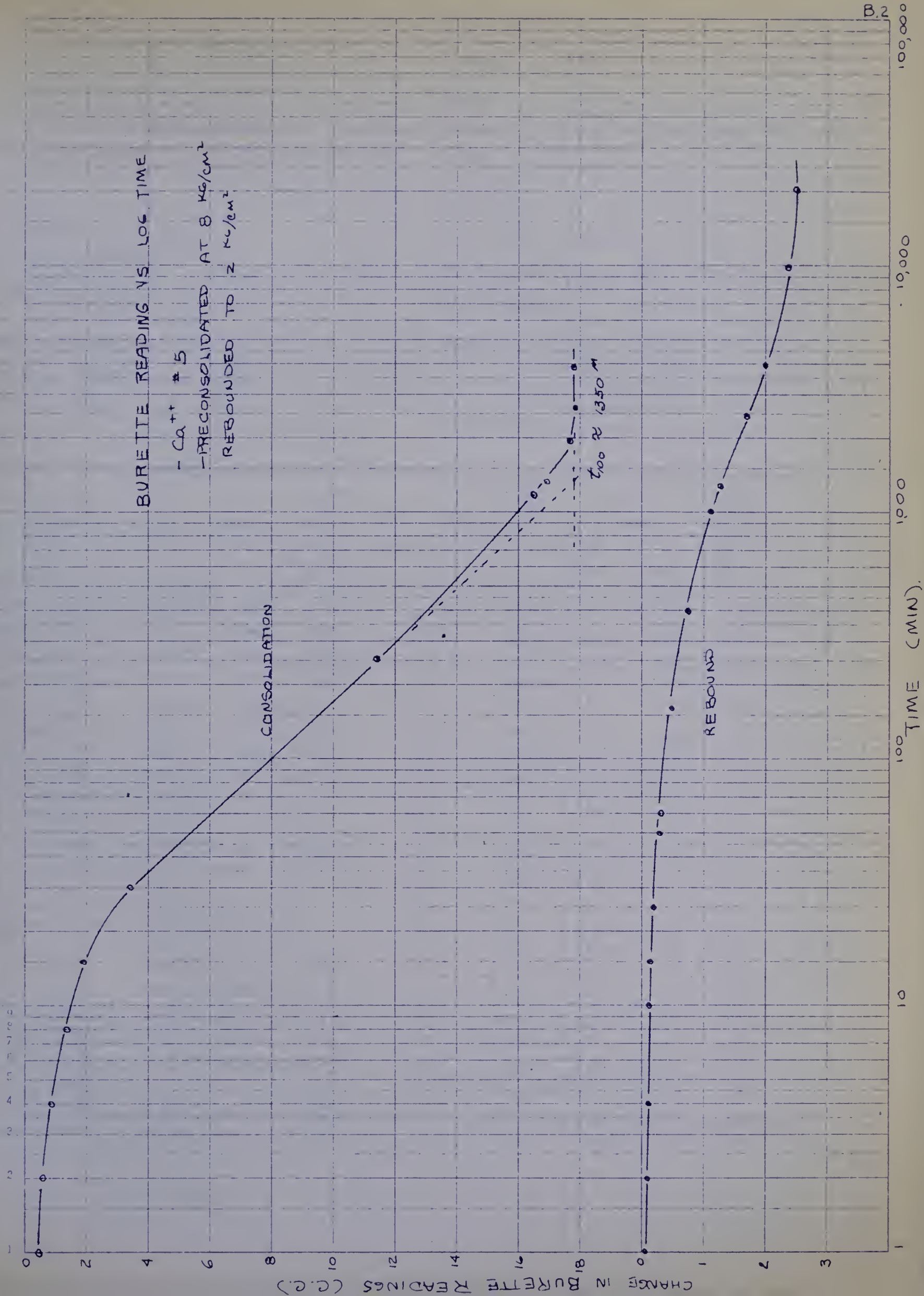
$$= \frac{1}{3} (A_T + 2A_C + A_B)$$

$$= 10.189 \text{ cm}^2$$

REBOUND

~~CONSOLIDATION~~ DATA - cont'd2 KG/CM² - (1 STEP)

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
FEB 20/67			
15:55	0	17.80	0.00
	15 ^s	17.84	0.04
	30 ^s	17.86	0.06
	1 ^m	17.87	0.07
	2	17.89	0.09
	4	17.90	0.10
	10	17.92	0.12
	15	17.95	0.15
16:20	25	18.00	0.20
16:45	50	18.08	0.28
16:55	60	18.10	0.30
18:35	160	18.28	0.48
22:30	395	18.55	0.75
FEB 21			
8:40	1005	18.93	1.13
13:30	1295	19.07	1.27
FEB 22			
8:45	2450	19.48	1.68
FEB 23			
10:00	3965	19.80	2.00
FEB 27			
13:35	9940	20.17	2.37
MAR 6			
15:50	20160	20.30	2.50



UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

TRIAXIAL COMPRESSION TEST ON COHESIVE SOIL

 Project THESIS

Hole No. _____

Depth _____

 Sample # 5 - Ca⁺⁺

 Engineer G. GILCHRIST

Technician _____

 Date of Test MARCH 8, 1967

 Test Lateral Pressure $\sigma_3 = 4 \text{ kg/cm}^2$ $\tau_3' = 2 \text{ kg/cm}^2$

 Back Pressure 2 kg/cm²

 Remarks RATE OF STRAIN 4394 min/10 mm.
 $\tau_3' = 2.00 \text{ kg/cm}^2$
 $\tau_3' = 2.40 \text{ kg/cm}^2$

 VOL. INDICATOR + 2.8 \odot

 Area Correction Factor 1.155

Data at Failure

$(\sigma_1 - \sigma_3) = 7.34 (1.20) 1.91 \sigma_1 = 3.91 (4.01)$

$P = .96 (1.20) 1.91 (1.61)$

$\bar{\sigma}_1 = 2.41 (2.00) 2.00 (2.40)$

$\bar{\sigma}_3 = 1.04 (0.80) .09 (.39)$

Strain 7.1 (7.6) 2.3 (2.3)%

Time min	Strain Dial Div.	A _c cm ²	No. of Stress Dial Div.	O. 3248 Proving Ring Const kg/Div 30.6	$\sigma_1 - \sigma_3$ $= \frac{k_p \cdot \delta \cdot K}{A_c}$	8.958 x 10 ⁻⁷ Pore Press kg/cm ² P _p 1116.3	Effective Stress		Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{\bar{A}}{P_p}$ $\frac{P_p}{\sigma_1 - \sigma_3}$
							$\bar{\sigma}_1$ Major	$\bar{\sigma}_3$ Minor			
MARCH 8											
13:08	.100	10.1892	11700	0.00	0.00	19970	2.00	2.00	1.00	0.00	
13:45	.108	10.1993	11495	205	.7588	19215	2.083	1.324	1.57	0.10	
14:25	.116	10.2096	11380	320	1.183	18650	1.99	.809	2.46	0.20	
14:50	.172	10.2817	11190	510	1.872	18340	1.96	.540	4.46	0.90	
18:13	.176	10.2874	11188	512	1.879	18330	1.97	.531	4.54	0.95	
18:30	.180	10.2921	11182	518	1.900	18280	1.91	.486	4.90	1.00	
21:22	.204	10.3234	11182	518	1.894	17870	1.88	.119	16.89	1.30	
22:03	.212	10.3338	11181	519	1.896	17860	1.89	.110	18.24	1.40	
MAR 9											
1:45	.260	10.3971	11149	551	2.001	18190	1.60	.405	5.94	2.00	
2:22	.268	10.4077	11148	552	2.002	18180	1.603	.397	6.05	2.10	
4:00	.288	10.4344	11172	528	1.91	17840	1.91	.09	21.76	2.35	
4:20	.292	10.4397	11173	527	1.91	17840	1.91	.09	21.73	2.40	
10:00	.364	10.5367	11152	548	1.96	18220	1.57	.432	5.54	3.30	
10:40	.372	10.5498	11150	540	1.93	18220	1.57	.432	5.45	3.40	
13:45	.412	10.6027	11202	498	1.77	17990	1.77	.23	8.85	3.90	
14:25	.420	10.6137	11212	488	1.74	18010	1.73	.27	7.40	4.00	
19:25	.484	10.7029	11258	412	1.56	18710	1.13	.87	2.79	4.80	
20:02	.492	10.7141	11260	440	1.55	18700	1.14	.86	2.80	4.90	
21:18	.508	10.7367	11298	402	1.41	18400	1.41	.59	3.40	5.10	
21:55	.512	10.7424	11301	399	1.40	18400	1.41	.59	3.36	5.15	
MAR. 10											
9:20	.660	10.9561	11300	400	1.38	18080	.98	2.40	2.35	7.0	
9:57	.668	10.9679	11302	398	1.37	19990	.96	2.41	2.32	7.1	
13:10	.708	11.0272	11350	350	1.20	18630	1.20	.80	2.50	7.6	

DEVIAIOR STRESS VS AXIAL
COMPRESSIVE STRAIN

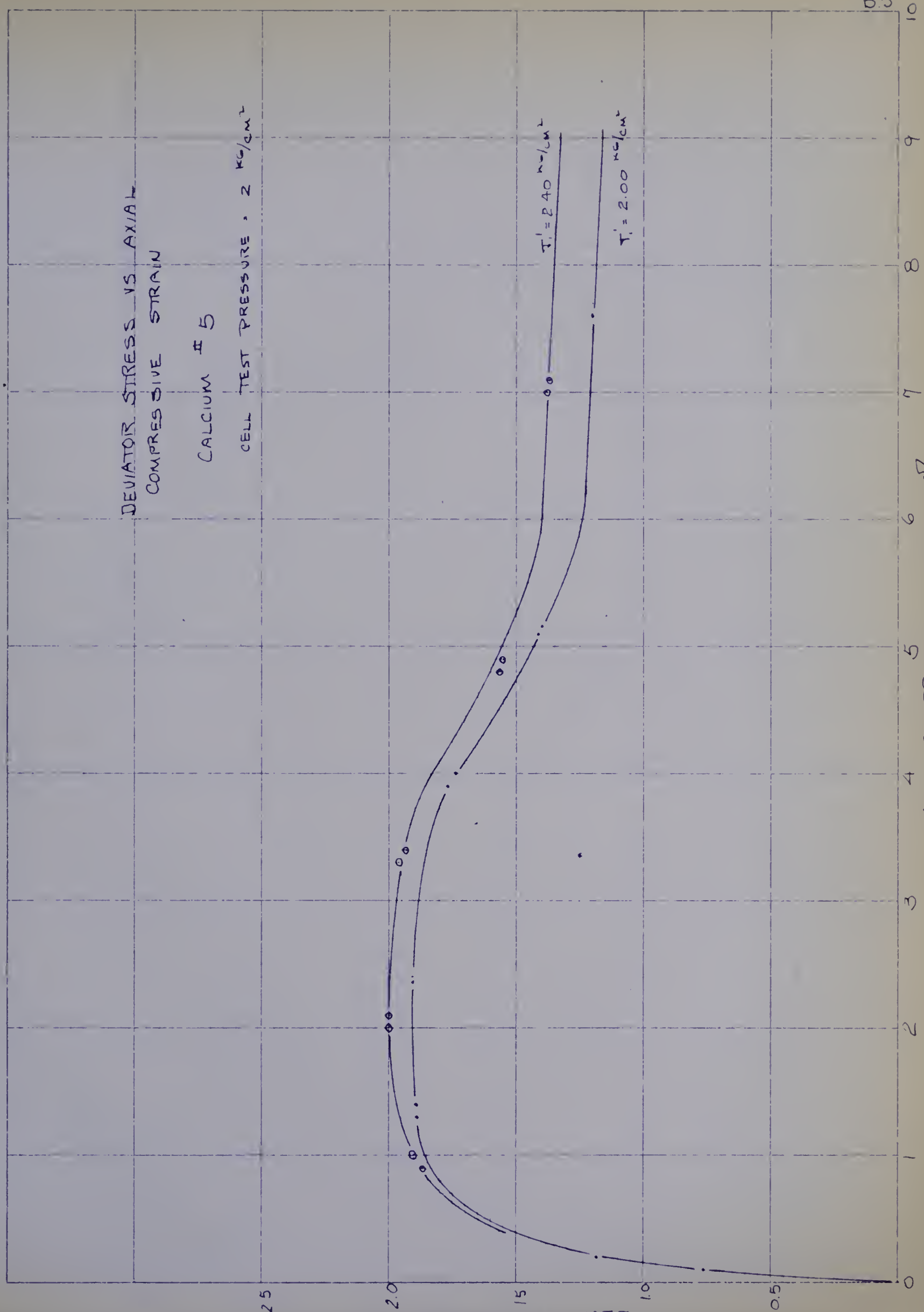
CALCIUM # 5

CELL TEST PRESSURE = 2 KG/CM²

DEVIAIOR STRESS (KG/CM²)

AXIAL COMPRESSIVE STRAIN %

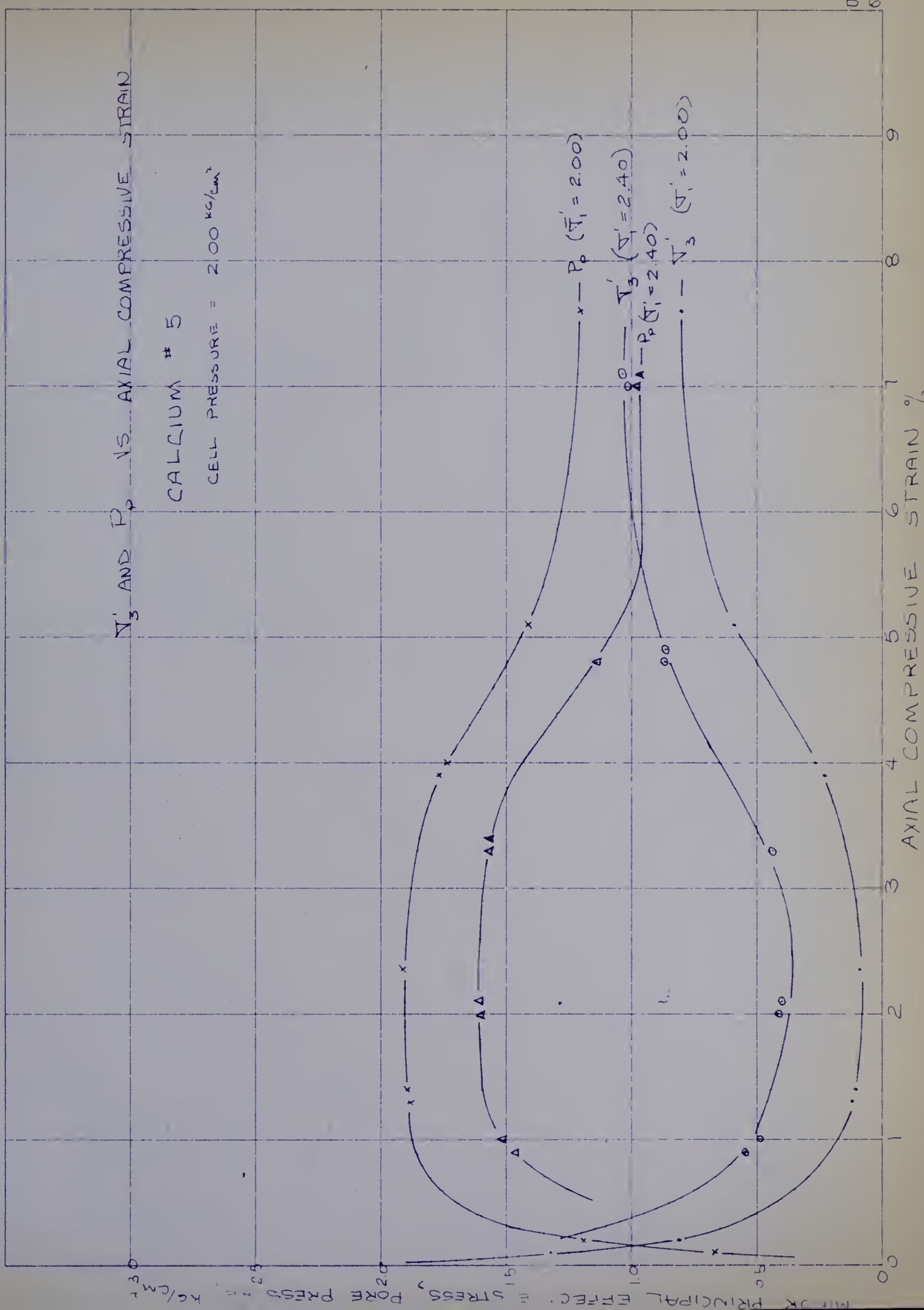
B5

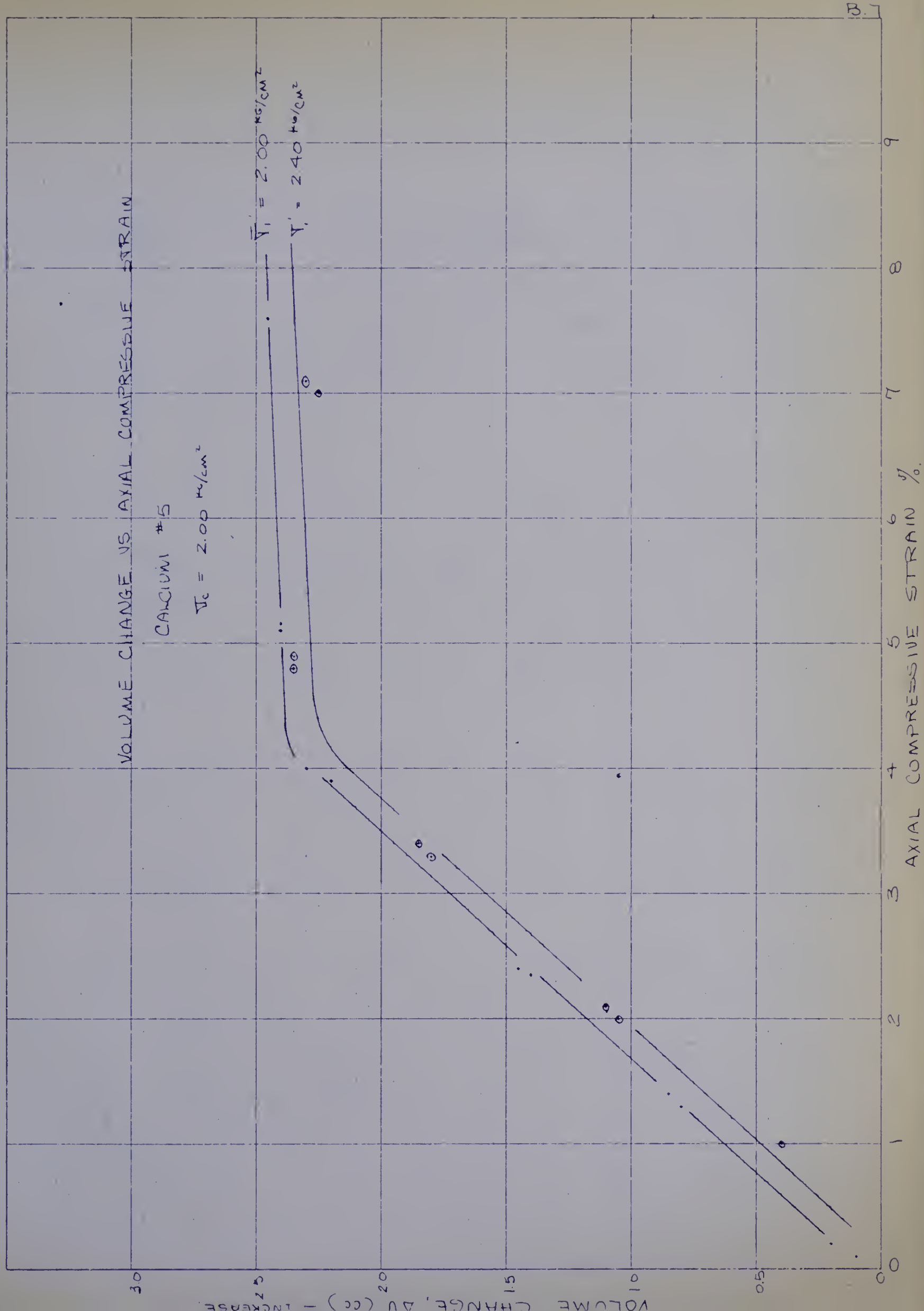


σ'_3 AND P_p VS. AXIAL COMPRESSIVE STRAIN

CALCIUM # 5

CELL PRESSURE = 2100 kg/cm²





CALCULATION OF ϕ_e AND C_e CALCIUM NO. 5 - $\bar{V}_c = 200 \text{ kg/cm}^3$

$E \%$	\bar{V}_1 kg/cm ³	$\Delta \bar{V}_1$ kg/cm ³	\bar{V}_d kg/cm ³	$\frac{\bar{V}_d}{Z}$ kg/cm ³	$\Delta \bar{V}_d$ kg/cm ³	$\sin \phi_e = \frac{\Delta \bar{V}_d}{2\Delta \bar{V}_1 - \Delta \bar{V}_d}$	ϕ_e°	$\cos \phi_e$	$C_e = \frac{\frac{\bar{V}_d}{Z} - (\bar{V}_1 - \frac{\bar{V}_d}{Z}) \sin \phi_e}{\cos \phi_e}$
.6	2.40	.40	1.71	.855	.04	.053	3.0	.999	.775
	2.00		1.67						
.8	2.40	.40	1.82	.91	.05	.057	3.8	.998	.813
	2.00		1.77						
1.0	2.40	.40	1.90	.95	.06	.081	4.65	.997	.835
	2.00		1.84						
2.0	2.40	.40	2.00	1.00	.09	.127	7.30	.992	.829
	2.00		1.91						
3.0	2.40	.40	1.98	.99	.09	.127	7.30	.992	.818
	2.00		1.81						
4.0	2.40	.40	1.89	.92	.10	.143	8.2	.990	.717
	2.00		1.74						
5.0	2.40	.40	1.56	.78	.13	.194	11.2	.981	.479
	2.00		1.43						
6.0	2.40	.40	1.40	.70	.16	.250	14.5	.968	.284
	2.00		1.24						
7.0	2.40	.40	1.38	.69	.17	.270	15.7	.963	.239
	2.00		1.21						
8.0	2.40	.40	1.35	.67	.17	.270	15.7	.963	.208
	2.00		1.18						

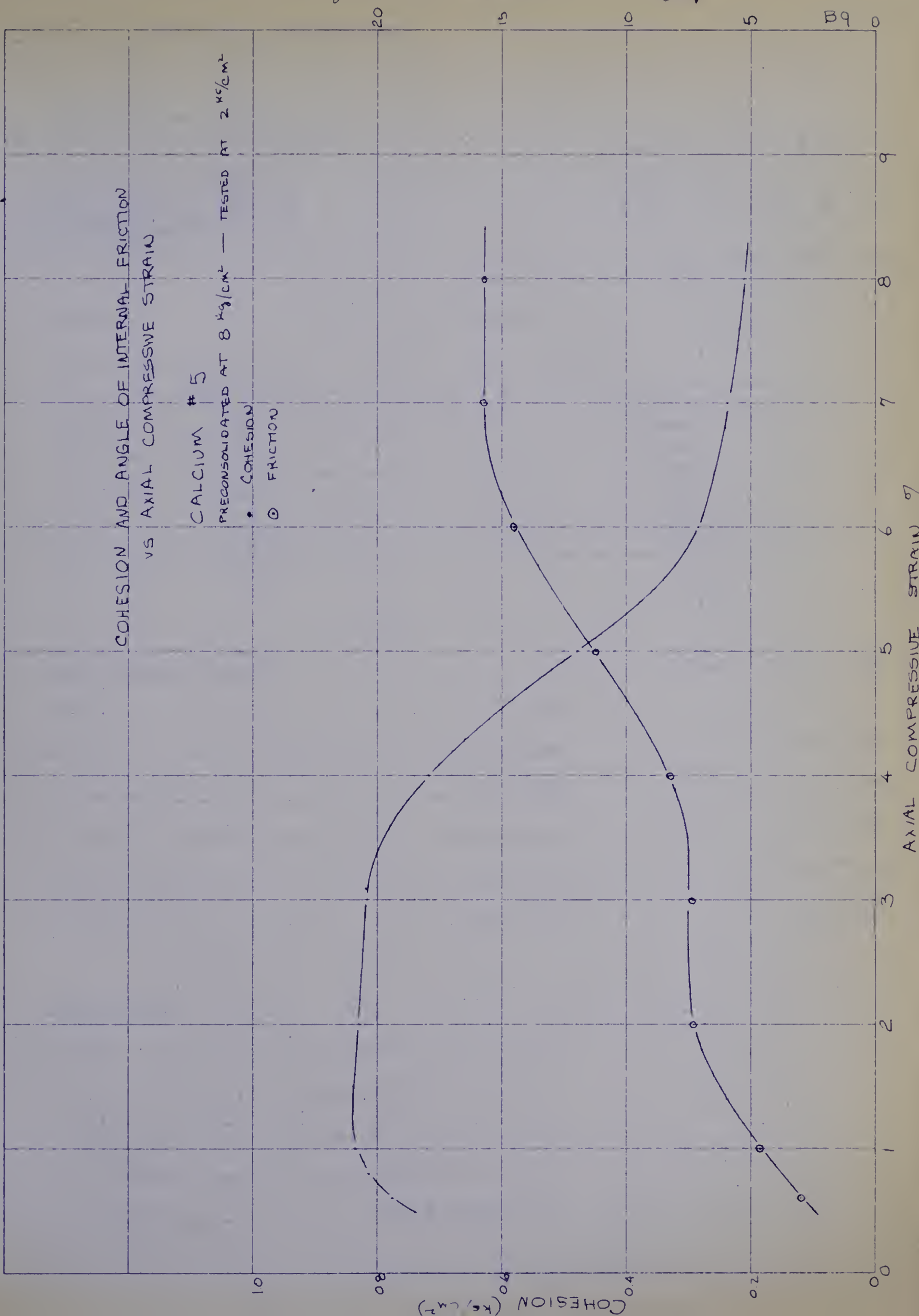
COHESION AND ANGLE OF INTERNAL FRICTION
VS AXIAL COMPRESSIVE STRAIN

CALCIUM # 5

PRECONSOLIDATED AT 8 kg/cm² — TESTED AT 2 kg/cm²

• COHESION

○ FRICTION



UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING

SOIL MECHANICS LABORATORY
TRIAxIAL COMPRESSION TEST
PORE PRESSURE REACTION TEST

Sample No. #5 Ca⁺⁺

Sample Desc. _____

Kg/sq. cm on cell 2 1

1 Kg/sq cm = 1116.3 MICRO INCHES/INCH.

"LOAD"

t min	Pp Kg/cm ²	(MICRO-INCHES/INCH)	Pp
0		20,000	
0.1		19,080	
0.25		19,850	
0.50		19,820	
1.0		19,770	
2.		19,680	
3		19,620	
4		19,570	
5		19,540	
8		19,450	
10		19410	.529

$\approx 53\%$

"UNLOAD"

t min	Pp Kg/cm ²
0	19410
0.1	
0.25	19,600
0.50	19,650
1.0	19,700
2.	19,760
3	19,810
4	19,840
5	19,870
8	19,920
10	19,945

Final Moisture Content

Wet Wt. plus tare 77.18 gms.
 Dry Wt. plus tare 66.39 gms.
 Wt. of water 10.79 gms.
 Wt. and No. of tare 33.62 gms.
 Wt. of dry soil 32.77 gms.
 Final M.C. 32.93 %

INITIAL MOISTURE CONTENT.

112.99 gms.
 100.92 gms.
 12.07 gms.
 74.57 gms.
 26.35 gms.
 45.8 %

Final Volume

Wt. Hg + Tare 1004.12
 Tare 90.20
 Wt. Hg 914.52
 Temp. 24.2°C
 Vol. Hg 67.56 cc.

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING

SOIL MECHANICS LABORATORY
TRIAXIAL COMPRESSION TEST COMPUTATIONS

Sample No. #5 Ca⁺⁺
Sample Desc. _____

BEGINNING OF TEST

Original Vol of Specimen 81.58 cc.
Wt. of Soil Solids in Specimen 97.78 gms.
Vol. of Soil Solids 35.95 cc.
Vol of Voids 45.63 cc.
Original Void Ratio, e_o 1.27
Original Porosity, n , 0.56%
Wt. of Water 44.79 gms.
Original Degree of Saturation, S , 98.0%
Original Wet Wt., lb/cu. ft. _____
Original Dry Wt., lb/cu. ft. _____

END OF TEST

Final Vol of Specimen (by Hg immersion) 67.56 cc.
Vol. of Soil Solids 35.57 cc.
Vol. of Voids 31.99 cc.
Final Void Ratio, e_f 0.90
Final Porosity, n , 47%
Wt. of Water 31.86 gms.
Final Degree of Saturation 99.7%
Final Wet Wt. lb/cu.ft. _____
Final Dry Wt. lb/cu.ft. _____
Wet wt. of specimen at beginning of test 142.57 gm.
Wet wt. of specimen at end of test 128.62 gm.
Weight loss 13.95 gm.
Vol. change from burette rdg. discrepancy 12.85 cc.
Discrepancy 1.10 cc



APPENDIX C

SUMMARY OF TEST DATA

TABLE C.1
SUMMARY OF TEST DATA

TEST NO	SOIL TYPE	γ_g kg/cm ²	γ_p kg/cm ²	O.C.R.	G_s	CONS. TIME 200 MIN	SECOND CONS. TIME MIN	REBOUND TIME MIN.	PORE PRESSURE REACTION %	MOISTURE CONTENT %			VOID RATIO		FINAL DEGREE OF SATUR. %	RATE OF STRAIN - MIN. FOR 1 %	STRAIN AT END OF TEST %	AT MAX. DEVIATOR	
										INITIAL	AFTER CONS.	AFTER REBOUND	INITIAL	FINAL				$\Delta \epsilon / \epsilon_p$ %	$\Delta \epsilon / \epsilon_p$ %
2	Na	8	1	8	2.75	100,000	—	—	—	58.6	25.7	—	1.63	—	—	—	—	—	—
4	Ca	8	1	8	2.72	510	3640	11,630	57	46.3	26.1	30.3	1.26	0.93	102.9	350	8.25	0.98 0.89	1.7
5	Ca	8	2	4	2.72	1410	2490	20,160	53	45.8	27.6	30.2	1.27	0.90	99.7	350	7.6	2.00 1.91	2.0
6	Ca	8	4	2	2.72	840	3060	LEAK	82	46.8	26.1	—	1.29	0.76	90.6	350	8.6	3.00 2.76	4.0
7	K	8	1	8	2.72	5200	9300	44,835	104	46.9	27.7	32.2	1.27	1.04	98.2	1280	10.2	1.07 1.00	2.0
10	Mg	8	1	8	2.69	1370	4540	9755	76	44.5	25.6	24.6	1.21	0.96	99.7	1280	10.7	1.00 0.82	1.2



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